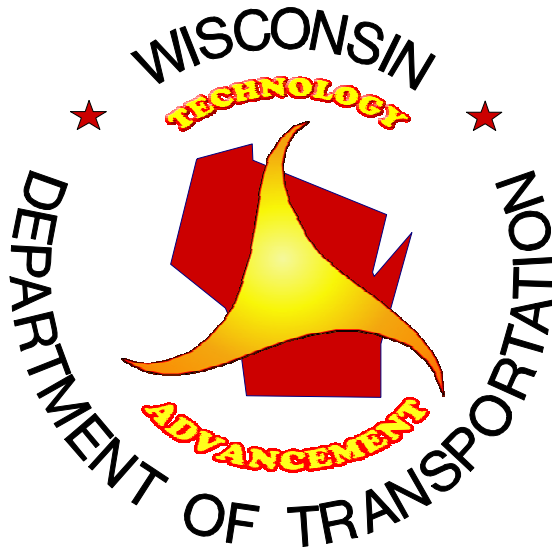


TIRE RUBBER IN HOT MIX ASPHALT PAVEMENTS

FINAL REPORT



MAY 2004

1. Report No. WI-06-02	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle Tire Rubber in Hot Mix Asphalt Pavements		5. Report Date May 2004	
		6. Performing Organization Code WisDOT Research Studies # WI-89-04 & 93-01a	
		8. Performing Organization Report No. WisDOT Report # WI-06-02	
7. Author(s) Deb Bischoff, WisDOT Technology Advancement Engineer Amanda Toepel, WisDOT Technology Advancement Engineer		10. Work Unit No. (TRAIS)	
9. Performing Organization Name and Address Wisconsin Department of Transportation DTID/BHC, Technology Advancement Unit 3502 Kinsman Blvd., Madison, WI 53704-2507		11. Contract or Grant No.	
		13. Type of Report and Period Covered Final Report, 1990-2004	
12. Sponsoring Agency Name and Address Wisconsin Department of Transportation Div of Trans Infrastructure Development, Bureau of Highway Construction Technology Advancement Unit 3502 Kinsman Blvd., Madison, WI 53704-2507		14. Sponsoring Agency Code WisDOT Research Studies # WI-89-04 & 93-01a	
		15. Supplementary Notes	
16. Abstract <p>In 1989 and 1993, WisDOT initiated two separate research studies to evaluate (1) the effectiveness of a rubberized asphalt binder mixed with a virgin aggregate gradation for use as an overlay and/or a stress absorbing interlayer, and (2) the recyclability of a reclaimed asphaltic pavement (RAP) containing tire rubber. These studies are presented in this report.</p> <p>Numerous test sections and control sections, of different pavement designs, were constructed for performance evaluations and comparisons. The results and conclusions of the studies are summarized below.</p> <ul style="list-style-type: none"> • The construction of the CRM HMA pavement and the recycled CRM RAP went fairly smoothly; only minor complications were encountered, primarily due to lack of experience with paving CRM asphaltic mixes, and were easily overcome • Emission data showed that recycling a pavement that contains tire rubber does not appear to pose a threat to the health of the workers or to the environment. • All test sections performed comparably with their respective control section, and no individual test section/pavement design clearly outperformed all others. <ul style="list-style-type: none"> ⇒ The inclusion of crumb rubber into a virgin HMA pavement for use as an overlay or as an interlayer did not enhance or impede the overall performance of the pavement, in comparison to a virgin HMA pavement. ⇒ The RAP mix containing tire rubber was successfully recycled and performed similarly to a new HMA pavement. ⇒ The asphalt rubber did not appear to increase or decrease the amount or rate of reflective cracking. • The inclusion of crumb rubber increased the cost of the rubberized asphalt binder by 76 to 271 percent, and the CRM mixes by 52 to 117 percent. A CRM HMA interlayer increased the project cost by approximately \$10,000 per lane-mile. • Cost savings were achieved when the CRM RAP was milled for recycling, as pavements constructed with reclaimed asphaltic pavement are generally less expensive than pavements constructed with virgin materials. <p>With performances similar to standard HMA pavements and higher costs, WisDOT's crumb rubber modified hot mix asphalt pavements have not proven to be cost-effective.</p>			
17. Key Words scrap tire, tire, rubber, CRM, crumb rubber, rubberized asphalt binder, RAP, asphalt, asphaltic concrete pavement, asphalt rubber, recycled, recycled asphalt pavement, interlayer, HMA		18. Distribution Statement DISTRIBUTION UNLIMITED AUTHORIZED FOR PUBLIC RELEASE	
19. Security Classification (of this report) UNCLASSIFIED	20. Security Classification (of this page) UNCLASSIFIED	21. No. of Pages 60	22. Price

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**WISCONSIN FEDERAL EXPERIMENTAL PROJECT # WI 89-04
and
WISDOT RESEARCH STUDY # 93-01a**

FINAL REPORT # WI-06-02

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MAY 2004

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INTRODUCTION

Finding alternative uses for previously landfilled materials has been a primary focus for many industries, including the transportation industry. Scrap tires, in particular, generate huge volumes of waste; nationally, approximately 280 million waste tires are generated annually. The disposal of scrap tires creates several direct problems. In addition to taking up a significant amount of landfill space, tires are difficult to bury as they cannot be easily compacted and they are too buoyant to remain covered. The alternative of stockpiling waste tires, however, poses immediate hazards. Tire stockpiles are ideal breeding grounds for mosquitoes, some of which carry deadly diseases like Encephalitis and the West Nile Virus. Tire stockpiles are also fire hazards. Tire fires are extremely difficult to extinguish and some have taken several days to put out. In an attempt to find alternative uses for scrap tires, the Wisconsin Department of Transportation (WisDOT) has conducted several research studies on possible uses of crumb rubber, finely ground tire chips no greater than 1/4-inch in any dimension, in highway construction applications.

In the past, crumb rubber modified (CRM) asphalt had been used with reclaimed asphaltic pavement (RAP) mixes in Wisconsin and the performance results were not promising. For instance, in 1987, WisDOT conducted a study on two different highways, USH 12 in Eau Claire County and STH 35 in Vernon County, to evaluate the use of rubberized asphalt binder in an overlay. One of the highway projects consisted of a 6-inch asphaltic concrete pavement that was milled down two to three inches. The milled material was salvaged and recycled for use in the new 3-inch overlay. The other highway project involved a variable thickness, asphaltic concrete (AC) pavement, overlying a PCC pavement. The scope of that project involved milling off 2½ inches of the existing AC pavement and recycling it for use in the new 3-inch overlay. Both highway projects included two test sections and one control section, consisting of a RAP with a CRM HMA interlayer test section, a full depth CRM RAP test section, and a full depth RAP control section. The test sections with a CRM HMA interlayer consisted of a lower course and surface course of RAP with a CRM HMA interlayer between the two courses. The full depth CRM RAP test sections were composed of rubberized asphalt binder mixed with RAP. Within the first two years of this study, the CRM RAP test sections developed approximately five times more transverse cracking than the standard RAP control sections. The RAP test sections with a CRM HMA interlayer performed about the same as the RAP control sections.

Despite the disappointing results from the 1987 study, WisDOT later conducted two additional research studies in a continuing effort to find effective methods of utilizing waste tires in highway construction applications. One study, WisDOT Federal Experimental Project (FEP) # WI-89-04, began in 1989 to evaluate the performance of rubberized asphalt binder mixed with a virgin aggregate gradation for use as an interlayer and/or an overlay. The second study, WisDOT Research Study # 93-01a, was initiated in 1993 with the objective of evaluating the recyclability of RAP pavements containing tire rubber. This report presents the results of these two studies.

BACKGROUND

It is estimated that approximately five million used tires are generated in the state of Wisconsin each year. Prior to 1988, scrap tires in Wisconsin were primarily stockpiled or sent to landfills. In order to reduce the amount of stockpiled tires in the state, the Wisconsin Waste Tire Removal and Recovery Program was initiated in 1988. The program provided funding, from a \$2 per tire fee on new vehicle registrations, to clean up existing stockpiles, to develop or expand existing and new markets/technologies for the reuse of waste tires, and to develop/operate a waste tire collection program to prevent the illegal stockpiling of waste tires. Due to this initiative, when the program ended in 1997, the amount of stockpiled scrap tires in the state was less than 1% of the original amount (roughly 15 million tires). The majority of those tires were used by industries and utilities for tire derived fuel, which continues to be the primary market in Wisconsin for utilizing scrap tires.

The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 mandated the use of recycled rubber in asphaltic pavements constructed in 1994 and thereafter. However, the standards provided in the “Use of Asphaltic Pavement Containing Recycled Rubber” portion of the Act caused many state and local transportation agencies to be apprehensive. Not only would the inclusion of recycled rubber increase the cost of the pavement, but also, any effects the rubber asphalt materials would have on the health of the workers were unknown and the performance results of asphaltic pavements containing recycled rubber were uncertain. In fact, several states, including Wisconsin, had evidence that pavements constructed in the 1980’s with recycled tire rubber performed equal to or poorer than conventional pavements, at 50% - 100% higher costs. A lack of stockpiled scrap tires also fueled Wisconsin’s opposition to the mandate.

Since taking a proactive approach to the mass amounts of tires being stockpiled in the state with the Wisconsin Waste Tire Removal and Recovery Program, the state's tire stockpiles were significantly reduced. In order to comply with the mandate, Wisconsin would have had to import scrap tires from other states. In 1995, the negative feedback from several states, including Wisconsin, resulted in the removal of the mandate for the use of recycled tires in asphaltic pavements.

Although energy production is still the primary market in Wisconsin for utilizing waste tires, environmental and economic factors have caused the fuel market for scrap tires to decline since 1997. With the funding from the Wisconsin Waste Tire Removal and Recovery Program depleted, there is little incentive for industries and utilities to continue to reuse tires for energy production. Thus, tires are again beginning to accumulate in stockpiles. Due to the declining fuel market for waste tires and a 1995 regulation banning scrap tires from landfills, it is imperative that other uses for waste tires be evaluated.

As previously mentioned, despite the disappointing results from prior studies, WisDOT conducted two additional research studies (WisDOT FEP # WI 89-04 and WisDOT Study # 93-01a) in a continuing effort to find effective methods of utilizing scrap tires in hot mix asphalt (HMA) pavements. These two studies are summarized in this report.

WISDOT FEDERAL EXPERIMENTAL PROJECT # WI 89-04

“Evaluation of Crumb Rubber Modified (CRM) Hot Mix Asphalt (HMA)”

PROJECT DESCRIPTION

This study was initiated in 1989 to evaluate the effectiveness of a rubberized asphalt binder mixed with a virgin aggregate gradation for use as an overlay and/or a stress absorbing interlayer.

The CRM HMA used for this study was prepared by first mixing the rubberized asphalt binder (often referred to as asphalt-rubber) utilizing a wet process. The rubberized asphalt binder

contained standard asphalt cement, an extender, finely ground scrap tire rubber, and a small portion of virgin rubber.

Specifically, the design of the rubberized asphalt binder was:

Koch 200/300 Asphalt Cement	--	81%
San Joaquin 1200S Extender	--	3%
Baker IGR-24 Rubber (scrap tire rubber)	--	13%
Tonson C112 Rubber (virgin rubber)	--	3%

The gradations of the rubber components are shown on pages 3 and 4 of Appendix A. Both types of rubber were delivered on pallets, in 50-pound bags. The virgin rubber and finely ground scrap tire rubber were blended with the extender and the 200 – 300 penetration grade asphalt cement at 325° - 375° F.

The CRM HMA **pavement** mix used for the surface and lower lifts of the overlay was completed by mixing a virgin aggregate gradation with 5.25% rubberized asphalt binder at 300° – 325° F. The CRM asphaltic pavement was field compacted at 280° F. The CRM HMA mix design is summarized below. See Appendix A for detailed design information.

Mix Design - Job Mix Formula (50 Blow Marshall) for STH 57

Aggregate:

Source: Sheboygan Sand & Gravel
 24% - 5/8" Coarse Aggregate
 76% - Crushed Fines

Asphalt Binder:

5.25 % Rubberized Asphalt Binder

Job Mix Formula Gradation: C-3

<u>Sieve</u>	<u>% Pass</u>	<u>Spec Range</u>
3/4 inch	100.0	100
1/2 inch	92.9	90 - 97
3/8 inch	80.8	78 - 92
No. 4	54.6	48 - 62
No. 8	38.5	29 - 41
No. 16	25.3	- - -
No. 30	15.3	14 - 24
No. 50	11.4	9 - 19
No. 200	5.4	3 - 7

Mix Properties:

Spec Limits

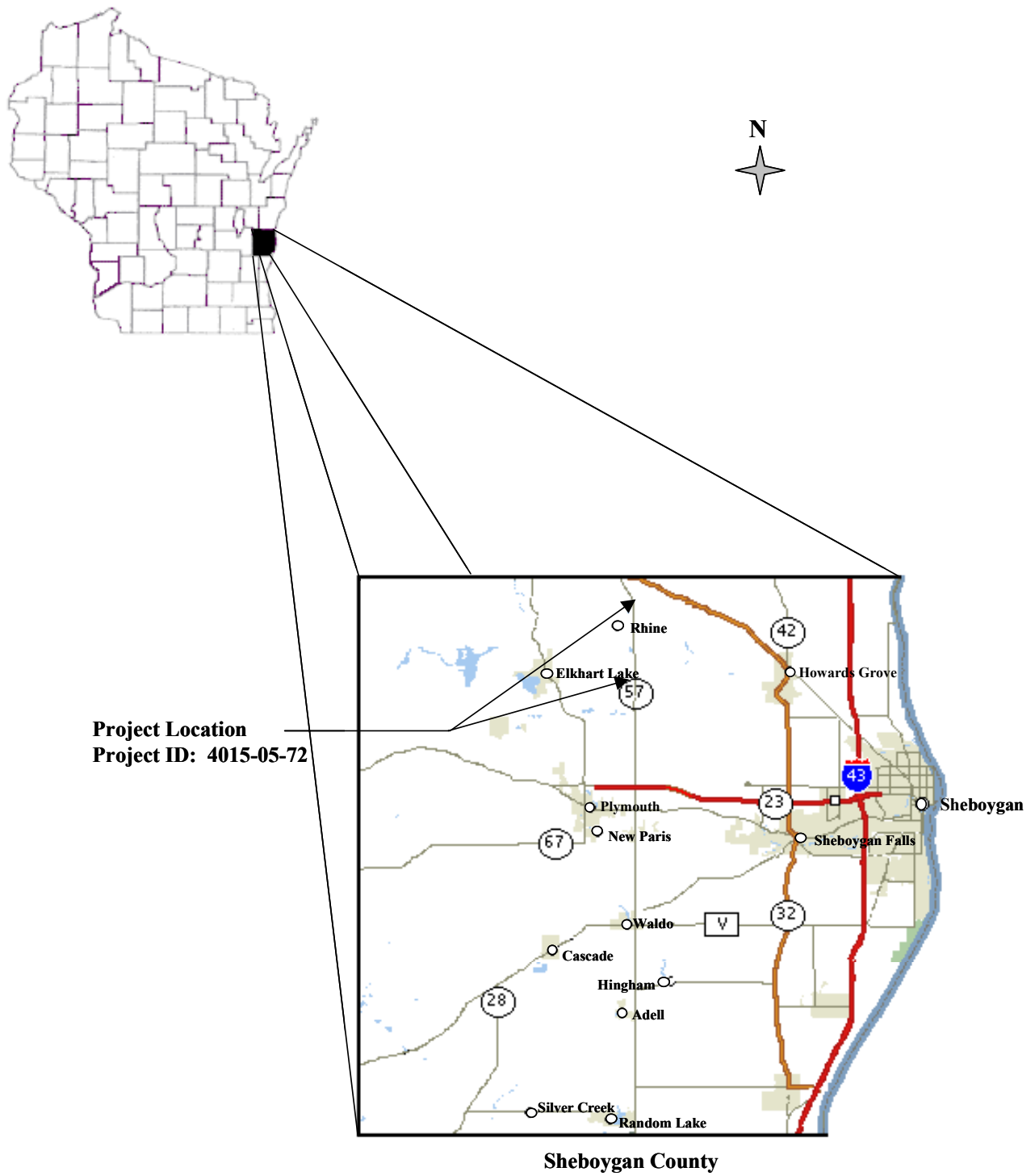
Air Voids:	3.0%	3 - 5
VMA:	14.2%	15.0 min (Gse)
Stability:	1836 lbs	1200 min
Flow:	16 (1/100 in)	8 - 18
TSR:	86.7%	70.0

The CRM HMA **interlayer** was constructed by applying the rubberized asphalt binder directly to the lower (bottom) course of the overlay, then immediately spreading precoated stone chips (precoated with standard asphalt binder) on the rubberized binder. Three rubber tire rollers followed behind, embedding the stone chips into the rubberized binder. Excess stones were broomed off prior to paving the surface course.

PROJECT LOCATION

The project site was located on State Trunk Highway (STH) 57 in Sheboygan County, Wisconsin, between Plymouth and the north county line. More specifically, the southern limit of the test site was located just north of Garton Road and the northern limit was located just south of County Trunk Highway (CTH) EH (see Figure 1, page 6).

Figure 1. CRM HMA Study Location



TEST SECTIONS

The rehabilitation of this two-lane rural highway consisted of overlaying an existing Portland Cement Concrete (PCC) pavement with a HMA pavement. The existing PCC pavement, originally constructed in 1956, was doweled and mesh reinforced, with 80-foot joint spacings.

Numerous test sections containing CRM HMA components (pavement and/or interlayer) were integrated into the 8.6-mile long rehabilitation project scope and were constructed for future performance evaluations and comparisons. Specifically, three different test sections and one control section were constructed and replicated in June of 1990, resulting in six test sections and two control sections. All of the sections were 2500 feet long, except for Control Section 1b, which was 2000 feet in length. The composition of the sections were as follows:

Test Section 1a: Station 400+00 to 425+00

2-inch Standard HMA Surface Course

$\frac{3}{8}$ -inch CRM HMA Interlayer

* $\frac{3}{4}$ -inch to 1½-inch Standard HMA Lower Course

Test Section 2a: Station 425+00 to 450+00

2-inch CRM HMA Surface Course

$\frac{3}{8}$ -inch CRM HMA Interlayer

* $\frac{3}{4}$ -inch to 1½-inch CRM HMA Lower Course

Test Section 3a: Station 450+00 to 475+00

1¼-inch CRM HMA Surface Course

* 1¼-inch to 2-inch CRM HMA Lower Course

Control Section 1a: Station 505+00 to 530+00

1¼-inch Standard HMA Surface Course

* 1¼-inch – 2-inch Standard HMA Lower Course

Test Section 1b: Station 530+00 to 555+00

2-inch Standard HMA Surface Course

$\frac{3}{8}$ -inch CRM HMA Interlayer

* $\frac{3}{4}$ -inch to $1\frac{1}{2}$ -inch Standard HMA Lower Course

Test Section 2b: Station 555+00 to 580+00

2-inch CRM HMA Surface Course

$\frac{3}{8}$ -inch CRM HMA Interlayer

* $\frac{3}{4}$ -inch to $1\frac{1}{2}$ -inch CRM HMA Lower Course

Test Section 3b: Station 580+00 to 605+00

$1\frac{1}{4}$ -inch CRM HMA Surface Course

* $1\frac{1}{4}$ -inch to 2-inch CRM HMA Lower Course

Control Section 1b: Station 621+00 to 641+00

$1\frac{1}{4}$ -inch Standard HMA Surface Course

* $1\frac{1}{4}$ -inch to 2-inch Standard HMA Lower Course

*The smaller thickness represents approximate overlay thickness at the outside edge of pavement and the larger thickness represents approximate overlay thickness at the pavement centerline.

CONSTRUCTION OBSERVATIONS

Northeast Asphalt, Inc., from Green Bay, Wisconsin, was the primary contractor for this project. International Surfacing, Inc., from Chandler, Arizona, was the subcontractor and supplied the equipment, manpower, and expertise for the blending of the rubber mix, in addition to applying the rubberized asphalt binder of the interlayer to the lower (bottom) course of the overlay prior to the placement of the stone chips. They also monitored the paving operation, sieve analysis, gradation testing, and density testing.

According to the November 5, 1990, construction report prepared by Philip DeCabooter and John Steinhauer, titled, "Asphalt-Rubber Pavement Report", "Prior to paving, bad areas of base

were removed and replaced with asphaltic base patching; cracks and joints were cleaned and filled with tar sealer; frost heave areas were undercut and one section was raised 1.5 feet above the existing grade; culverts were replaced; intersections were widened; and other work incidental to this type of a project was done.”

According to the project engineer, some complications were encountered during construction of both the CRM HMA pavement and the CRM HMA interlayer, the majority of which were due to inexperience with paving crumb rubber modified hot mix asphalt.

The biggest problem encountered during construction of the CRM HMA pavement was that the roller operators had to work much harder than usual to achieve the specified densities. Identifying a rolling pattern that would work consistently well for the length of the project proved to be a difficult task. If the initial vibratory roller made extra passes to achieve density, sometimes the tar sealer, used to fill the underlying concrete pavement’s joints, would seep up through the overlay. It was determined that the higher heat of the CRM HMA mix caused the tar sealer to liquefy underneath the overlay and seep upward through the rubberized HMA during compaction.

Other difficulties encountered while working with the CRM HMA included: a stickier consistency than a standard HMA, causing buildup of the mix on the workers’ shoes and paving equipment and requiring more time than usual for cleaning the shoes and equipment; negative attitudes from the truck drivers regarding a special provision banning the use of diesel fuel (and suggesting the use of a soap/water solution or silicone emulsion) to coat their truck boxes; the appearance of scattered, small tears behind the paver that could not be explained (that disappeared after the first pass of a roller); and a longer set time than standard HMA, requiring extra care to prevent the pavement from breaking off along the shoulder edges. It was also noted that the CRM HMA had a stronger and more offensive odor, and provided a blacker surface, than a standard asphaltic mix.

The paving operation of the CRM HMA interlayer ran fairly smooth, but two additional problems were observed. When the rubberized asphalt binder was applied directly to the first

(lower) course of the overlay, a lot of blue smoke and steam was created. At times, the smoke was so thick that it was difficult to see paving equipment from fifty feet away. The contractor also experienced some uneven spreading of the stone chips in one area of the project. It was determined that the high heat of the rubberized asphalt binder heated up the tires of the stone chip spreader, resulting in a washboard effect with the stone chips. Releasing air from the spreader's tires resolved the problem.

PERFORMANCE EVALUATION

The overall performance of each CRM HMA test section was evaluated and compared against the control sections using five main parameters: (1) crack surveys; (2) rut depth measurements; (3) pavement surface distress (using the Pavement Distress Index, or PDI); (4) ride measurements (using the International Roughness Index, or IRI); and (5) friction measurements.

1. Crack Surveys

Since the use of rubber was believed to reduce the amount of reflective cracking, the percentage or amount of reflective cracking was monitored. Thus, 500-foot segments were identified in each test and control section, and monitored for reflective cracking throughout the study period. All joints and cracks of the existing concrete pavement were documented and each succeeding survey (after construction of the asphaltic overlay) compared the number of new cracks that had developed to the original number of joints and cracks prior to the overlay.

Crack surveys were conducted annually over the first five years, and then again after ten years. A final crack survey was also conducted twelve years after construction. The results of these surveys are shown in Table 1, page 11. It should be noted that some of the cracks that developed after the overlay was placed were probably thermal or fatigue cracks and not necessarily reflective cracks. Since the pavement was monitored from the standpoint of overall performance, no attempt was made to distinguish between reflective, thermal, or fatigue cracking, with all cracks considered reflective cracks. The individual crack counts were used to determine the percentage of reflective cracking in each test and control section. Tables 2 and 3, on pages 11 and 12 respectively, show the percentage of reflective cracking per test section and per pavement design, respectively.

Table 1. Number of Cracks Per Test Section

	Test 1a	Test 2a	Test 3a	Control 1a	Test 1b	Test 2b	Test 3b	Control 1b
Initial	68	66	75	70	58	78	70	76
1st year	37	44	41	29	22	29	40	39
2nd year	42	45	51	37	43	46	47	51
3rd year	47	48	56	40	47	48	55	52
4th year	59	51	58	47	51	53	57	65
5th year	59	52	59	49	52	60	58	66
10th year	60	55	59	56	55	67	60	70
12th year	60	56	59	58	55	69	60	71

Table 2. Percent Reflective Cracking Per Test Section

	Test 1a	Test 2a	Test 3a	Control 1a	Test 1b	Test 2b	Test 3b	Control 1b
1st year	54	67	55	41	38	37	57	51
2nd year	62	68	68	53	74	59	67	67
3rd year	69	73	75	57	81	62	79	68
4th year	87	77	77	67	88	68	81	86
5th year	87	79	79	70	90	77	83	87
10th year	88	83	79	80	95	86	86	92
12th year	88	85	79	83	95	88	86	93

Test 1a & 1b = Standard HMA Surface & Lower Course with CRM HMA Interlayer

Test 2a & 2b = CRM HMA Surface & Lower Course with CRM HMA Interlayer

Test 3a & 3b = CRM HMA Surface & Lower Course - No Interlayer

Control 1a & 1b = Standard HMA Surface & Lower Course - No Interlayer

Table 2 shows that Control Section 1a (standard HMA overlay with no interlayer) had the lowest percentage of reflective cracking from the second year through the fifth year, and ranked second best after ten and twelve years in service. Test Section 3a (CRM HMA surface and lower course with no interlayer) had the lowest amount of reflective cracking after ten and twelve years in service. Its replicate section, Test Section 3b, ranked fairly average in regards to reflective cracking throughout the study duration. Test Section 1b (standard HMA surface and lower course with a CRM HMA interlayer) started out well and had the second lowest percentage of reflective cracking after the first year. By the second year, however, and through the remainder of the study, Test Section 1b had the highest percentage of reflective cracking. Its replicate

section (Test Section 1a) also ranked poorly, in regards to percentage of reflective cracking, after the fourth year and through the remainder of the study period.

Table 2 shows no clear trend regarding best performing pavement type. In fact, the results of each test section are inconsistent with its replicate section, indicating that other factors, such as underlying pavement condition and subgrade, may have had a larger impact in the amount of reflective cracking than the overlay design.

Table 3. Average % Reflective Cracking per Pavement Design

Pavement Design	Year After Construction						
	1	2	3	4	5	10	12
Test Sections 1a & 1b: Standard HMA Surface & Lower Course with CRM HMA Interlayer	46	68	75	87	88	92	92
Test Sections 2a & 2b: CRM HMA Surface & Lower Course with CRM HMA Interlayer	52	64	67	73	78	85	87
Table Sections 3a & 3b: CRM HMA Surface & Lower Course - No Interlayer	56	68	77	79	81	82	82
Control Sections 1a & 1b: Standard HMA Surface & Lower Course - No Interlayer	46	60	63	76	78	86	88

Table 3 shows the average percentages of reflective cracking of the four different pavement designs (standard HMA overlay with a CRM HMA interlayer, CRM HMA overlay with a CRM HMA interlayer, CRM HMA overlay with no interlayer, and standard HMA overlay with no interlayer). These values were calculated by simply averaging the percentage of reflective cracks of each test section and its replicate section. During the first three years, the standard HMA overlay with no interlayer (Control Sections 1a and 1b) averaged the least amount of reflective cracking, while the CRM HMA overlay with no interlayer (Test Sections 3a and 3b) averaged the most reflective cracking. By the fourth year, and through the twelfth year, the standard HMA overlay with a CRM HMA interlayer (Test Sections 1a and 1b) performed the worst, averaging five to twelve percent more reflective cracking than the other pavement designs after twelve years. Despite the fact that Test Sections 3a and 3b (CRM HMA overlay with no interlayer) averaged the highest percentage of reflective cracking over the first three years, they averaged

six to twelve percent less reflective cracking than the other pavement designs after twelve years in service.

The crack surveys revealed that there was no noticeable difference in the physical characteristics of the transverse cracks between the different test sections and the control sections. During the first three years following construction, the majority of all the cracks were severity level 1 (less than ½-inch in width). However, by the fourth year, some severity level 2 cracks (greater than ½-inch in width) did develop. The maintenance crew of the county highway department routed and sealed the cracks shortly after the four-year crack survey was taken, making it difficult to determine the severity levels of the transverse cracks during subsequent crack surveys; thus, all of the transverse cracks were reported as severity level 1. Longitudinal cracking was also observed at the centerline joint and at the outer pavement edge throughout most of the project length. Longitudinal cracking within the lanes was minimal in all test sections.

Overall, no individual test section or pavement design clearly outperformed all others. Thus, the introduction of rubber into the pavement mix does not appear to increase or decrease the amount or rate of reflective cracking.

2. Rut Depth Measurements

Repetitive loadings in the wheel paths of a flexible pavement, such as asphalt, can cause the formation of ruts (surface depressions). Rut depth measurements were taken annually in the right wheel path of the northbound and southbound lanes of the test sections and control sections. After five years, the average rutting depths in all sections were very minimal, less than 0.05 inches. After ten years in service, the rutting didn't increase much; all rut depths were less than 0.1 inches, with an average of 0.06 inches after ten years in service. Thus, all test and control sections performed very well in terms of rut resistance.

3. Pavement Surface Distress

The Pavement Distress Index (PDI) is a unitless value that suggests overall pavement distress (i.e. cracking, rutting, raveling, etc.) based on visual inspections of the roadway. WisDOT's PDI values range from 0 to 100, with lower numbers indicating a pavement with less distress, and a

PDI value of about 70 or higher indicating a pavement that is a candidate for maintenance operations. It should be noted that all of the transverse cracks within the project limits were routed and sealed as part of routine maintenance work after the 1994 (fourth year) PDI evaluation. Since routing and sealing can hide the true severity of cracks, and crack severity impacts the PDI values, subsequent PDI values may have consequently been reduced.

Pavement distress surveys were conducted every other year throughout the study period. The PDI values of each test and control section are shown in Table 4 below.

Table 4. Average Pavement Distress Index (PDI) per Test Section

TEST SECTIONS	Years After Construction						
	0	2	4	6	8	10	12
Test Section 1a	0	28	37	28	28	28	28
Test Section 2a	0	23	33	23	23	28	28
Test Section 3a	0	23	28	28	28	28	28
Control Section 1a	0	23	44	28	28	28	28
Test Section 1b	0	28	32	23	23	28	28
Test Section 2b	0	23	38	28	28	28	28
Test Section 3b	0	23	28	23	23	28	28
Control Section 1b	0	23	28	13	13	28	28

While the first two distress surveys resulted in similar PDI values for all test sections, the fourth year inspection showed that Control Section 1a (standard HMA overlay with no interlayer) performed worse than the others, with a PDI value of 44. By the sixth year, after routing and sealing of the joints, most of the PDI values decreased, resulting in PDI values ranging between 13 and 28. The sixth and eighth year inspections showed that Control Section 1b 1a (standard HMA overlay with no interlayer) had the least amount of distress, with a PDI of only 13. After ten years in service, all test sections and control sections performed equally, with PDI values of 28. By 2002, the PDI values of all sections remained at 28, indicating that all sections continued to perform relatively well after twelve years in service.

Table 5. Average Pavement Distress Index (PDI) per Pavement Design

TEST SECTIONS	Years After Construction						
	0	2	4	6	8	10	12
Test Sections 1a & 1b: Standard HMA Surface & Lower Course with CRM HMA Interlayer	0	28	35	26	26	28	28
Test Sections 2a & 2b: CRM HMA Surface & Lower Course with CRM HMA Interlayer	0	23	36	26	26	28	28
Test Sections 3a & 3b: CRM HMA Surface & Lower Course - No Interlayer	0	23	28	26	26	28	28
Control Sections 1a & 1b: Standard HMA Surface & Lower Course - No Interlayer	0	23	36	21	21	28	28

Table 5 shows the average PDI of each pavement design. Throughout the study period, all pavement designs performed relatively equal in terms of pavement distress. Thus, based on the data shown in Tables 4 and 5, the rubberized asphalt components of the pavement didn't have much impact on overall pavement performance.

4. Ride Measurements

The International Roughness Index (IRI) is a ride quality measurement based on pavement roughness. IRI values are determined with a road profiler and recorded in the standard units of meters/kilometer. An increasing IRI value indicates an increase in the roughness of the ride. In accordance with WisDOT's standards, IRI values from 0 to 1.5 indicate a smooth ride typical of a new pavement, while values of about 2.6 or higher indicate a rough ride.

Ride data was collected annually for five years, from 1994 through 1998, and then every other year through 2002. The ride data was collected in both the northbound and the southbound directions. For each test and control section, the average IRI, between the northbound and southbound lanes, was determined (see Table 6, page 16). Table 7, on page 16, shows the average IRI value for each different pavement design.

Table 6. Average International Ride Index (IRI) per Test Section (m/km)

TEST SECTIONS	Years After Construction						
	4	5	6	7	8	10	12
Test Section 1a	0.91	0.96	0.91	0.94	1.03	1.06	1.04
Test Section 2a	0.80	0.94	0.91	1.07	1.09	1.06	1.04
Test Section 3a	1.22	1.25	1.23	1.31	1.35	1.28	1.42
Control Section 1a	0.92	0.97	1.05	1.08	1.12	1.06	1.07
Test Section 1b	0.93	1.09	0.94	0.93	1.07	1.06	1.07
Test Section 2b	0.95	1.04	1.04	1.06	1.17	1.22	1.22
Test Section 3b	1.07	1.14	1.09	1.14	1.18	1.22	1.22
Control Section 1b	0.96	1.12	1.12	1.16	1.25	1.22	1.22

Table 7. Average International Ride Index (IRI) per Pavement Design (m/km)

TEST SECTIONS	Years After Construction						
	4	5	6	7	8	10	12
Test Sections 1a & 1b: Standard HMA Surface & Lower Course with CRM HMA Interlayer	0.92	1.03	0.93	0.94	1.05	1.06	1.06
Test Sections 2a & 2b: CRM HMA Surface & Lower Course with CRM HMA Interlayer	0.88	0.99	0.98	1.07	1.13	1.14	1.13
Test Sections 3a & 3b: CRM HMA Surface & Lower Course - No Interlayer	1.15	1.20	1.16	1.23	1.27	1.25	1.32
Control Sections 1a & 1b: Standard HMA Surface & Lower Course - No Interlayer	0.94	1.05	1.09	1.12	1.19	1.14	1.15

Year four baseline IRI data shows that Test Section 3a (CRM HMA overlay with no interlayer) had the highest IRI after the first ride survey. The ride of Test Section 3a did not deteriorate much over the duration of the study, but consistently remained slightly rougher than all other sections. This indicates that the slightly rougher ride could be the result of other factors and is not necessarily due to the pavement type. Table 7 shows that, on average, the CRM HMA overlay with no interlayer had a 13 to 20 percent rougher ride than the other pavement designs after twelve years in service, while the standard HMA surface with a CRM HMA interlayer provided the smoothest ride. Nonetheless, after twelve years in service all of the test and control sections had IRI values below 1.5, reflecting a good, quality ride.

5. Friction Measurements

A high friction value is desired, as it is an indication of high skid resistance and a safe roadway. Basically, friction numbers of 35 or more are considered desirable for roadways. Friction data was collected in 1992 and 1993, two and three years after construction, respectively. The friction measurements were tested with ribbed tires at 40 miles per hour in the northbound and the southbound lanes of all six test sections and both of the control sections. The average friction numbers of each pavement design are shown in Table 8 below. Each pavement design provided a surface with good friction, with friction numbers ranging from 52 to 55.

Table 8. Average Friction Numbers @ 40 mph per Pavement Design

TEST SECTIONS	Years After Construction	
	2	3
Test Sections 1a & 1b: Standard HMA Surface & Lower Course with CRM HMA Interlayer	55.0	53.5
Test Sections 2a & 2b: CRM HMA Surface & Lower Course with CRM HMA Interlayer	52.0	52.0
Test Sections 3a & 3b: CRM HMA Surface & Lower Course - No Interlayer	53.5	53.0
Control Sections 1a & 1b: Standard HMA Surface & Lower Course - No Interlayer	55.0	54.0

Since the surface course of a pavement is primarily responsible for the frictional characteristics of the pavement, the friction numbers of the two surface types (standard HMA and recycled CRM RAP) were averaged and are shown in Table 9 below.

Table 9. Average Friction Numbers @ 40 mph Per Surface Composition

TEST SECTIONS	Years After Construction	
	2	3
Test Sections 1a & 1b and Control Sections 1a & 1b: Standard HMA Surface Course	55.0	53.8
Test Sections 2a, 2b, 3a, and 3b: CRM HMA Surface Course	52.8	52.5

Overall, the standard HMA surface course provided slightly more skid resistance than the CRM HMA surface course; the difference, however, was minimal. The inclusion of rubber in the HMA did not play a significant factor in the frictional characteristics of the pavement.

COSTS

The inclusion of scrap tire rubber in the HMA only affected the cost of materials for the project. For this particular project, the material mixing and placement costs were the same (\$8.96/ton), but the cost of the rubberized asphalt binder was 3.7 times more expensive than standard asphalt binder (\$455.00/ton vs. \$122.50/ton). With a 5.5 percent binder content, the total cost of the CRM HMA mixture was over twice as expensive as the standard HMA mix (\$33.99/ton vs. \$15.70/ton). The optional CRM HMA interlayer, composed of both the CRM HMA binder and the stone chips, also only affected the project cost. The total cost of the CRM HMA interlayer was \$440.00 per ton for the rubberized asphalt binder and \$21.25 per ton for the stone chips. Thus, with a CRM HMA binder application rate of 0.50-0.60 gal/yd² and a stone chip application rate of 30-40 lb/yd², the total cost of one lane-mile (12-foot wide) of the CRM HMA interlayer was approximately \$10,000.

RESULTS FOR WISDOT FEDERAL EXPERIMENTAL PROJECT # WI 89-04

This research study successfully incorporated scrap tire rubber, with a small portion of virgin rubber, into a HMA pavement by blending the rubber directly into the binder. The rubberized asphalt binder was then mixed with virgin aggregate and placed on an existing PCC pavement as a CRM HMA overlay, or sprayed on the lower lift of the overlay and covered with stone chips to form a CRM HMA interlayer. The interlayer was paved over with the surface course of the CRM HMA overlay. The main conclusions of this study are summarized as follows:

1. There were only minor complications encountered during the construction process, due in large part to the lack of experience with paving CRM HMA. The primary problem was that it was difficult to determine an effective rolling pattern to compact the CRM HMA material to the required density. Difficulties were also experienced in some areas due to the high temperature of the CRM HMA overlay. The high heat of the material created thick blue smoke and steam, caused the tar joint sealer of the underlying pavement to liquefy and bleed through the overlay, and heated up the tires of the stone chip spreader causing it to spread the stone chips unevenly.

2. The rubberized asphalt components of the pavement did not appear to increase or decrease the amount or rate of reflective cracking.
3. The rut depth measurements, pavement distress index (PDI), international roughness index (IRI), and friction data showed that all of the sections were in fairly good condition and performed comparably over the duration of the study.
4. The rubberized asphalt binder was 3.7 times more expensive than the standard asphalt binder; thus, the total cost of the CRM HMA pavement was over twice as expensive as the standard HMA pavement. The inclusion of a CRM HMA interlayer increased the cost of the pavement about \$10,000 per lane-mile.
5. No individual test section or pavement design clearly outperformed all others. Thus, the introduction of rubber into the asphalt binder, for use as an overlay or an interlayer, did not play a factor in overall pavement performance.

WISDOT RESEARCH STUDY # 93-01a

“Recyclability of Crumb Rubber Modified (CRM) Reclaimed Asphaltic Pavement (RAP)”

PROJECT DESCRIPTION

This study was initiated in 1993 to evaluate the recyclability of a reclaimed asphaltic pavement containing tire rubber.

BACKGROUND

As mentioned in the introduction of this report, in 1987, WisDOT conducted research studies on STH 35 in Vernon County and USH 12 in Eau Claire County to evaluate crumb rubber modified (CRM) reclaimed asphaltic pavement (RAP) material used as interlayers and/or overlays.

The construction of the test sections on STH 35 involved milling off two to three inches of the existing 6-inch HMA pavement surface, which was originally constructed in 1966. This milled material (which contained 95 percent crushed limestone, 5 percent sand, and 85-100 penetration grade asphalt cement added at a rate of 6.2 percent by weight) was blended with rubberized asphalt binder and new aggregate to form the CRM RAP mixture. The milled asphaltic pavement was then overlaid with the CRM RAP material.

The rubberized asphalt binder was composed of asphalt cement and crumb rubber (finely ground tire chips no greater than 1/4-inch in any dimension). The binder was produced using the wet process, by blending the crumb rubber with 200-300 penetration grade asphalt cement for a minimum of 30 minutes in a special blending unit before metering it into, and mixing it with, the RAP. All the components were mixed together at a temperature range of 300° - 325° F. The resulting mix consisted of 0.8 percent crumb rubber (15.5 lbs of rubber per ton of HMA mix).

The sub-contractor for the asphalt rubber work was International Surfacing, Inc., of Chandler, Arizona. The mix design for the crumb rubber modified mix was prepared by Crafcro, Inc., and is summarized on the following page.

1987 CRM RAP ON STH 35

Mix Design Job Mix Formula (50 Blow Marshall)

Reclaimed Asphaltic Pavement: 35%

95% Limestone (Zogg Quarry, Vernon Co)
5% Blend Sand (Krause Pit, Vernon Co)
6.2% 85-100 Asphalt Cement

Virgin Aggregate: 65%

70% Dolomitic Stone
30% Sand

Rubberized Asphalt Binder: 6.5%

4.3% Added Rubberized Binder
82% Koch 200/300 Asphalt Cement
18% Baker GR-24 Rubber (scrap tire rubber)
2.2% Residual Binder from RAP

JMF Gradation: 401.2.5 3-SP

<u>Sieve</u>	<u>% Pass</u>	<u>Spec Range</u>
3/4-inch	100.0	100
1/2-inch	98	95 - 100
3/8-inch	88	75 - 100
No. 4	63	45 - 85
No. 8	50	30 - 60
No. 16	---	---
No. 30	---	---
No. 50	21	10 - 30
No. 200	9.6	5 - 9

Core Tested Mix Properties:Spec Limits

Air Voids (%):	3.7	2-6
Flow (0.01 inch):	10	18 max
Density (lb/ft ³):	146.4	
Tensile Strength (psi)	192	

Two years after construction, the CRM RAP test sections on both the STH 35 and USH 12 projects showed a much greater frequency and severity of transverse cracking than the control sections. Lab test results indicated that the inclusion of rubber in the CRM RAP mixture produced a stiffer material than the standard RAP mixture, which contributed to the early cracking. An inadequate amount of rubberized asphalt binder, the use of a stiffer asphalt cement than typically used with rubberized asphaltic mixes, and/or variations in the rubber raw materials may also have contributed to the negative results.

In 1993, the CRM RAP pavement on STH 35, in Vernon County, was scheduled for rehabilitation. Thus, this study (WisDOT Research Study # 93-01a) was initiated to evaluate the recyclability of CRM RAP.

1993 RECYCLED CRM RAP PROJECT

In July of 1993, a 3200-foot long section of the CRM RAP on STH 35 was milled a depth of two inches. The salvaged CRM RAP millings were hauled off the project site and mixed with

approximately 80 percent virgin aggregate and 5.5 percent, 120-150 penetration grade, asphalt binder at 270° - 300° F for 1½ minutes. The resulting reprocessed asphaltic mixture, containing 20 percent CRM RAP and 0.15 percent rubber by total mixture weight, was used to construct several test and control sections. A summary of the mix design is shown below. See Appendix B for additional mix design information.

1993 Mix Design Job Mix Formula (50 Blow Marshall) for STH 35

CRM Reclaimed Asphaltic Pavement: 20%

65% Virgin Aggregate
35% RAP (1966)
6.5% Asphalt Binder

Virgin Aggregate: 80%

45% - Limestone (Pedretti Pit, Vernon Co)
25% - Blend Sand (Franz Pit, Crawford Co)
30% - Blend Sand (Prairie Pit, Crawford Co)

Asphalt Binder: 5.5%

4.4% Added 120/150 Koch Binder
1.1% Residual Binder from Rubberized RAP

JMF Gradation: MV - 3

<u>Sieve</u>	<u>% Pass</u>	<u>Spec Range</u>
3/4-inch	100.0	100
1/2-inch	93.5	90 - 100
3/8-inch	79.1	75 - 95
No. 4	61.8	45 - 80
No. 8	55.3	30 - 60
No. 16	49.1	- - -
No. 30	40.0	15 - 40
No. 50	20.6	10 - 25
No. 200	4.1	3 - 8

Mix Properties:

		<u>Spec Limits</u>
Air Voids (%):	3.4	3.5
VMA:	15.0	15.0 min (Gsb)
Stability:	1438	1200 min
Flow (0.01 in):	8.5	8 - 18
TSR:	95.4	70.0 min

PROJECT LOCATION

The 1993 project site was located on a two-lane rural segment of STH 35, approximately 12 miles south of the site where the CRM RAP millings were obtained. The limits of the resurfacing project began 1.2 miles south of the Vernon/Crawford County line and extended north to Genoa, in Vernon County. The test sections for this study began ¼-mile north of DeSoto and extended north to about one mile south of Victory (see Figure 2, page 24).

It should be noted that STH 35 is located adjacent to the Mississippi River, with steep embankments on both sides of the highway (see Photograph 1). The steep slopes and highly variable soils (talus material from the bluffs) may contribute to differential movements under extreme conditions, potentially affecting the pavement performance.



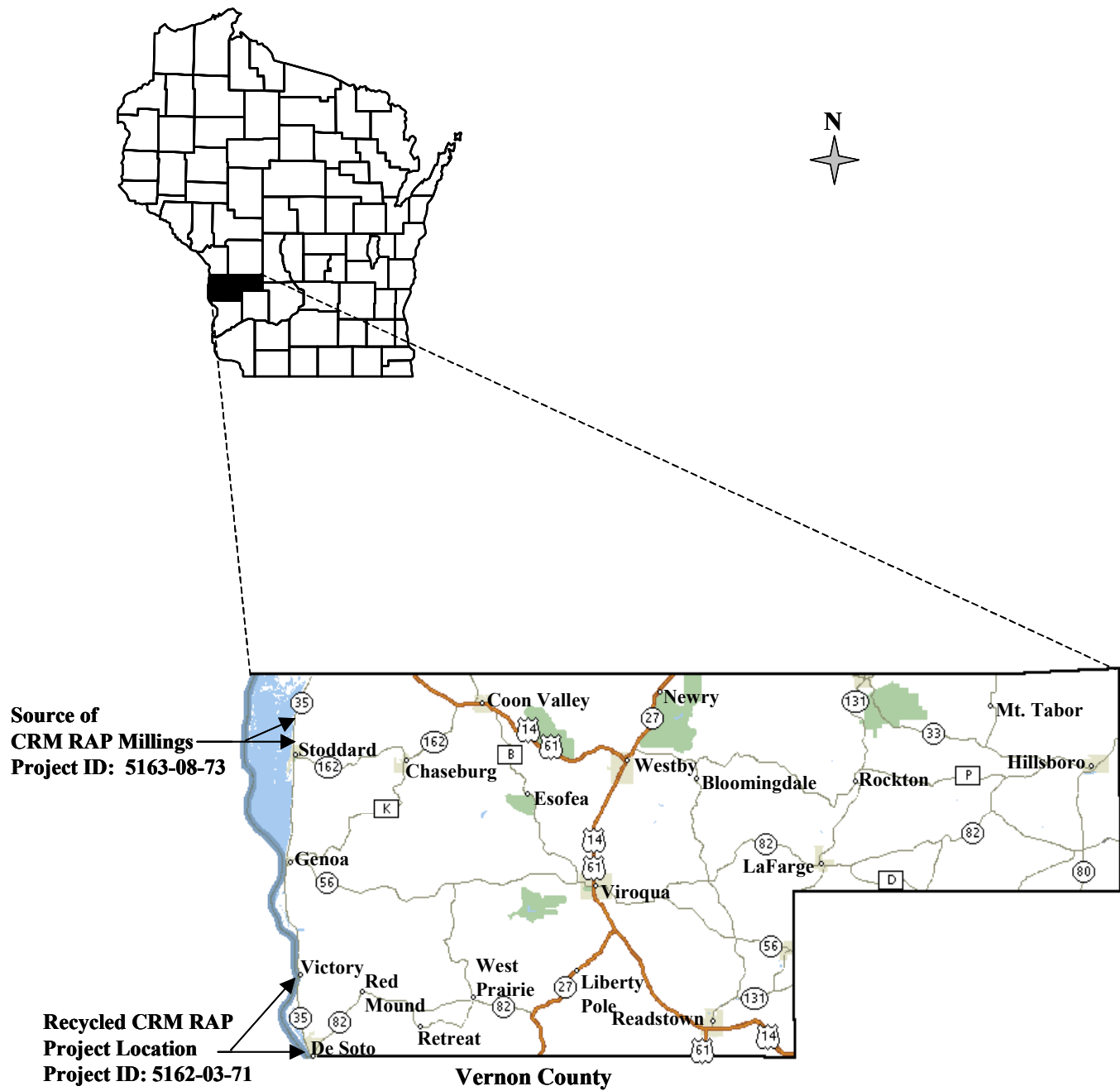
Photograph 1. STH 35 Recycled CRM RAP Project Site



Photograph 2. Slope Failure (South of Recycled CRM RAP Project Site)

In fact, in one extreme case in 2000, a segment of the western slope along STH 35, several miles south of the CRM RAP project site, failed, causing the partial loss of the southbound shoulder (see Photograph 2).

Figure 2. Recycled CRM RAP Study Location



TEST SECTIONS

The existing 3-inch HMA pavement, between DeSoto and Victory, was milled in-place and used as additional base course, over the existing 8-inch crushed aggregate base course and 12-inch granular subbase. Three test sections and one control section were constructed in 1993, for future performance evaluations and comparisons. Each test section was 2600 feet in length and consisted of the following:

Control Section: Station 80+00 to 106+00

1½-inch Virgin HMA Pavement, Type MV (Surface Course)

3-inch Virgin HMA Pavement, Type MV (Lower Course)

Test Section 1: Station 120+00 to 146+00

1½-inch Virgin HMA Pavement, Type MV (Surface Course)

3-inch Recycled CRM RAP Pavement, Type MV (Lower Course)

Test Section 2: Station 146+00 to 172+00

1½-inch Recycled CRM RAP Pavement, Type MV (Surface Course)

3-inch Recycled CRM RAP Pavement, Type MV (Lower Courses)

Test Section 3: Station 172+00 to 198+00

1½-inch Recycled CRM RAP Pavement, Type MV (Surface Course)

3-inch Virgin HMA Pavement, Type MV (Lower Course)

CONSTRUCTION OBSERVATIONS

Mathy Construction Co., from Onalaska, Wisconsin, was the primary contractor for this project. International Surfacing, Inc., from Chandler, Arizona, was the subcontractor and supplied the expertise for the blending of the rubber mix, monitoring of the paving operations, and testing of the CRM RAP material.

As stated in the construction and interim performance report written by Kurt Johnson, “Investigation of the Recyclability of Crumb Rubber Modified Asphaltic Concrete” (WisDOT

Report # WI/SPR-05-95), there were some minor differences between the CRM RAP material and the standard RAP mixtures. The milling operator found the rubberized material to be more resistant and harder to mill than a typical HMA pavement. This was expected, as earlier lab testing revealed that the CRM RAP material had a higher resilient modulus than the standard RAP mix without crumb rubber. Also, during the milling operation there was a distinct rubber odor; however, the odor was not strong enough to cause discomfort.

Standard paving and rolling operations were used and no complications occurred. Aside from the truck boxes requiring a coating of a release agent for each load, to keep the rubberized material from sticking, the rubberized RAP material responded in a manner similar to other standard RAP mixtures.

Emission testing was conducted at the plant during production of the paving material. Johnson noted that “from the analysis of emission data collected during the construction of the test sections there appears to be no increased health or environmental concerns with the recycling of rubber modified asphaltic pavement. The double drum asphaltic mix plant may have benefited production of lower emissions, but until tested against other plants this is not known.”

PERFORMANCE EVALUATION

The overall performances of all test and control sections were monitored through 2004. Each recycled CRM RAP test section was evaluated and compared against the control section using five main parameters: (1) rut depth measurements, (2) ride measurements (using the International Roughness Index, or IRI), (3) friction measurements, (4) site inspection, and (5) pavement surface distress (using the Pavement Distress Index, or PDI).

1. Rut Depth Measurements

Rutting, a form of distress unique to flexible pavements, is a permanent deformation in the pavement layers or subgrade. Generally, this distress is observed as longitudinal depressions in the wheel paths. Rut depths were measured over the length of each section using a road profiler. The data was then averaged over the distance of the section measured, to calculate the average depth of rutting in the right wheelpath. Rut depth measurements were collected annually for six

years and then every other year throughout the study duration. Rut depths were measured in the northbound and southbound lanes of each test/control section and the average values (to the nearest 0.01 inch) are shown in Table 10, below. Typically, rut depth measurements will stay the same or increase as a pavement ages. However, since the exact traveled path of the profiler may vary from one year to the next, it is possible that the average rut depth may decrease from the previous year.

Table 10. Average Rut Depth per Test Section (in.)

TEST SECTIONS	1994 (Year 1)	1995 (Year 2)	1996 (Year 3)	1997 (Year 4)	1998 (Year 5)	1999 (Year 6)	2001 (Year 8)	2003 (Year 10)
Control Section: Virgin HMA Surface & Lower Course	0.05	0.06	0.07	0.10	0.08	0.09	0.09	0.08
Test Section 1: Virgin HMA Surface Course & Recycled CRM RAP Lower Course	0.06	0.07	0.10	0.09	0.10	0.10	0.09	0.08
Test Section 2: Recycled CRM RAP Surface & Lower Course	0.06	0.07	0.07	0.09	0.09	0.07	0.05	0.07
Test Section 3: Recycled CRM RAP Surface Course & Virgin HMA Lower Course	0.04	0.06	0.07	0.10	0.09	0.08	0.08	0.08

All sections performed similarly throughout the study period, with average rutting depths of 0.07 or 0.08 inches after ten years in service. Thus, all sections performed well in terms rut resistance, as any rutting less than 0.25 inches is considered insignificant.

2. Ride Measurements

Ride quality, or smoothness, is measured with a road profiler, using the International Roughness Index (IRI). IRI is measured over the length of each section, and is reported in metric units of meters/kilometer (or in English units of inches/mile), with increasing values indicating an increase in the roughness of the ride. In accordance with WisDOT standards, IRI values from 0 to 1.5 indicate a smooth ride, while values of about 2.6 or higher indicate a rough ride. Since IRI values, like rut depth measurements, are measured using a road profiler, they are also sensitive to the exact path of the profiler. Although IRI values are expected to stay the same or increase as a pavement ages, if the profiler travels down a slightly different path than the previous year, the IRI values can decrease.

The IRI values were measured on STH 35 annually for five years after construction, with additional measurements taken in 2001 (year 8) and 2003 (year 10). The measurements were taken in the left wheel path of the northbound and southbound lanes of each section and averaged to the nearest 0.01 inch. The results are shown in Table 11 below.

Table 11. Average International Roughness Index (IRI) per Test Section (m/km)

TEST SECTIONS	1994 (Year 1)	1995 (Year 2)	1996 (Year 3)	1997 (Year 4)	1998 (Year 5)	2001 (Year 8)	2003 (Year 10)
Control Section: Virgin HMA Surface & Lower Course	0.88	0.86	0.83	0.86	1.02	0.98	1.12
Test Section 1: Virgin HMA Surface Course & Recycled CRM RAP Lower Course	0.89	0.88	0.85	0.95	1.05	1.13	1.35
Test Section 2: Recycled CRM RAP Surface & Lower Course	0.89	0.85	0.89	0.85	0.96	1.08	1.14
Test Section 3: Recycled CRM RAP Surface Course & Virgin HMA Lower Course	0.99	0.94	0.95	0.90	0.96	0.94	1.20

After ten years in service, Test Section 1 (virgin HMA surface course and recycled CRM RAP lower course) provided a slightly rougher ride than the other sections, with an IRI value of 1.35. However, all the ride values are less than 1.5 after ten years in service, indicating that all of the sections are still providing a smooth ride.

3. Friction Measurements

Friction numbers represent the skid resistance of a pavement, and typically, values of 35 or greater are desired for roadways. Friction data was collected three, five, and six years after construction. The friction measurements were tested in the northbound and southbound lane of all three test sections and the control section, at 40 miles per hour using ribbed tires. Friction is primarily affected by the composition of the surface course of a pavement; thus, the friction numbers were averaged for the test sections with the same surface course (i.e. Control Section & Test Section 1; and, Test Section 2 & Test Section 3). The average friction numbers for the two surface compositions are shown in Table 12 on the following page.

Table 12. Average Friction Numbers @ 40 mph Per Surface Composition

SURFACE COURSE COMPOSITION	1996 (Year 3)	1998 (Year 5)	1999 (Year 6)
Control Section and Test Section 1: Virgin HMA Surface Course	56.00	57.00	57.00
Test Section 2 and Test Section 3: Recycled CRM RAP Surface Course	55.00	55.00	54.50

The results revealed that each individual test section provided a surface with good skid resistance, with friction numbers consistently ranging from 54 to 57, three, five, and six years after construction. As shown in Table 12, the average friction values for both surface types were also very similar, with the recycled CRM RAP surface course providing slightly less skid resistance than the virgin HMA surface course. Overall, the use of the recycled rubberized RAP had little affect on the frictional characteristics of the roadway surface.

4. Site Inspection

A final site inspection, conducted in April 2004, showed that all four sections had a combination of Type I single (less than ½-inch in width) and Type III banded (multiple cracks in close proximity resulting in a narrow band of cracks) transverse and longitudinal cracks in both the northbound and the southbound lanes (see Photographs 3 & 4 below).



Photograph 3. Type III Banded Transverse Crack in the NB lane and Type I Single Transverse Crack in the SB lane

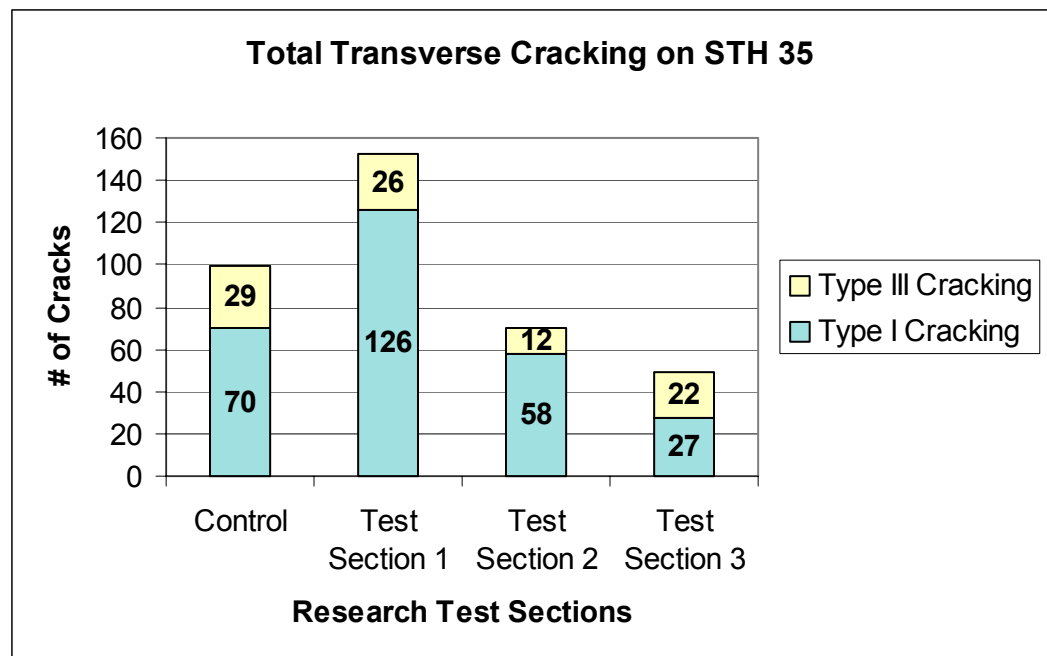


Photograph 4. Type III Banded Longitudinal Crack

There were, however, discrepancies in the performance within each test section. The southbound lane performed better than the northbound lane throughout the entire project length, with primarily Type I transverse cracking and less Type III longitudinal cracking. The performances of the individual sections also varied from one end to the other. For example, the northern half of the Control Section had primarily Type III banded transverse cracking in the northbound lane and Type I single cracks in the southbound lane, while the southern half of the same section consisted of primarily Type I single transverse cracks across both lanes. In addition, Test Section 3 had three isolated areas, totaling about one-third of the overall section length, that had primarily Type III cracking across the northbound and southbound lanes, while the remaining length of the section consisted of primarily Type I cracking across both lanes.

Overall, Test Section 1 (virgin HMA surface course and recycled CRM RAP lower course) had the highest number of transverse cracks, 17 percent of which were banded. Although Test Section 3 (recycled CRM RAP surface course and virgin HMA lower course) had the least amount of transverse cracks, it had the highest percentage of Type III banded transverse cracks, at 45 percent (see Figure 3 below).

Figure 3.



As previously mentioned, the southbound lane generally performed better than the northbound lane throughout the length of the test and control sections, with primarily Type I, and far less Type III, transverse cracking (see Figures 4 & 5, below). In fact, Test Section 2 didn't have any Type III transverse cracks, and Test Section 1 only had one, in the southbound lane.

Figure 4.

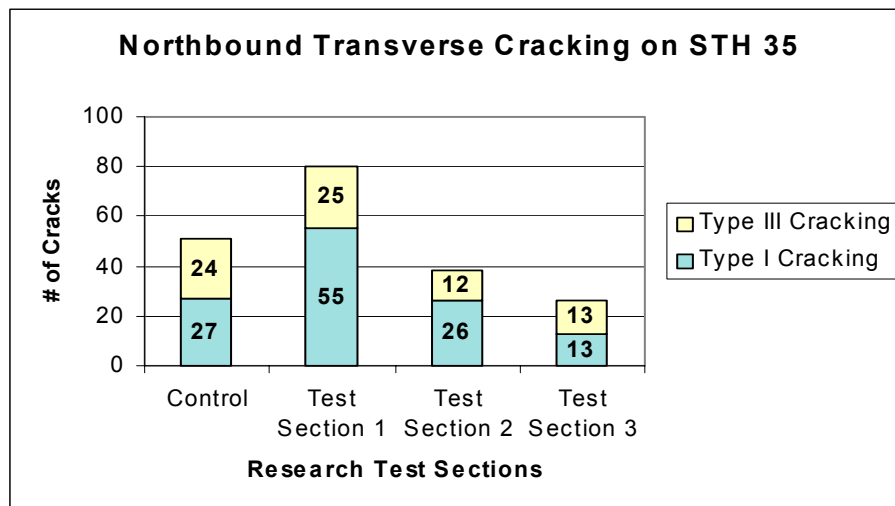
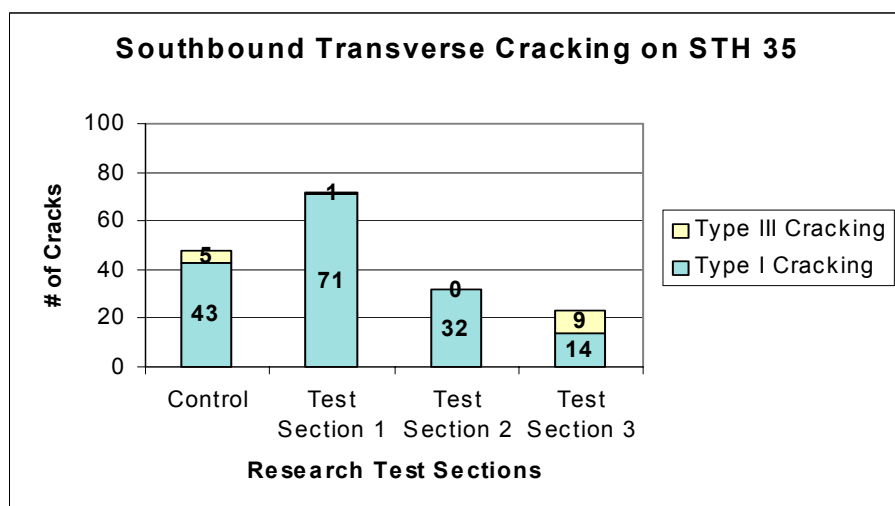


Figure 5.



The total linear footage of longitudinal cracking was measured in the northbound and southbound lanes of a 500-foot monitoring segment within each test section (see Figures 6-8). Figure 6 shows that the Control Section (virgin HMA surface and lower course) had the least amount of longitudinal cracking while Test Section 3 (recycled CRM RAP surface course and virgin HMA lower course) had the highest amount. Test Sections 1 & 2 (virgin HMA surface course and recycled CRM RAP lower course; and, recycled CRM RAP surface and lower course, respectively) had the least amount of Type III banded longitudinal cracking, particularly in the southbound lane (see Figures 7 & 8, page 33). Although the values are accurate for the 500-foot monitoring segments, they may not be truly representative of the longitudinal cracking present in the entire 2600-foot sections, as there were performance discrepancies within the test sections.

Figure 6.

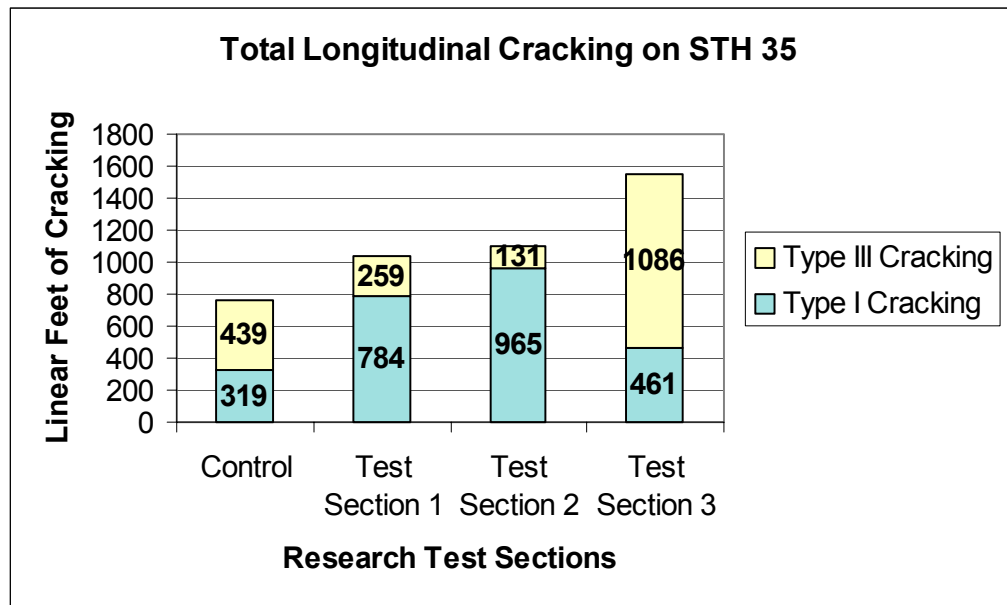


Figure 7.

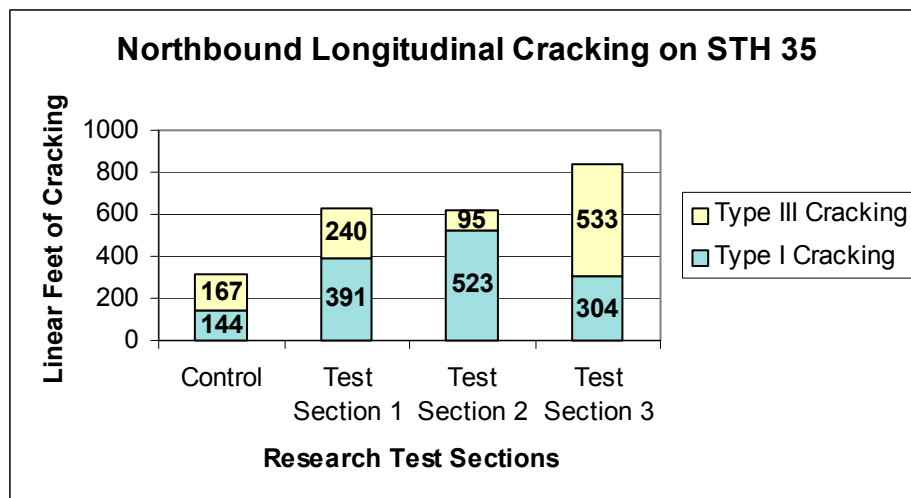
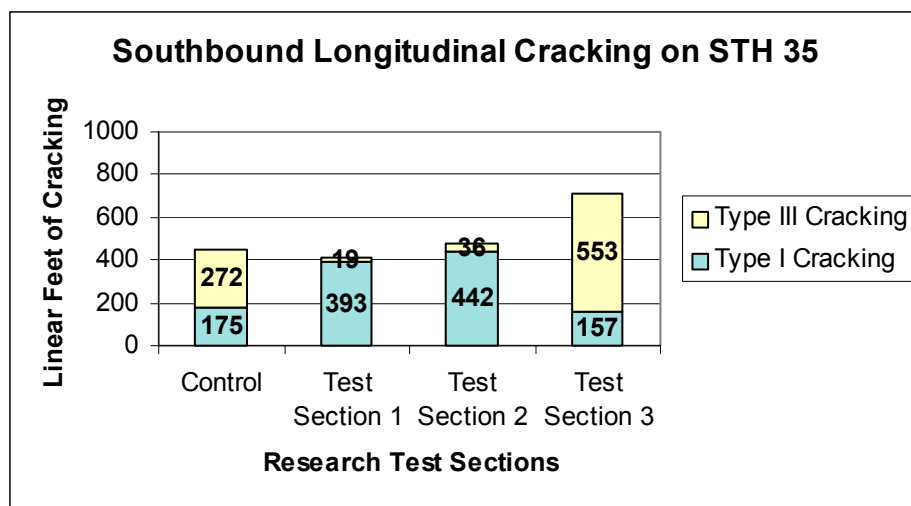


Figure 8.



5. Pavement Surface Distress

The Pavement Distress Index (PDI) values are unitless, and calculated from weighted factors that are applied to the amount, type, and severity of distress. WisDOT's PDI values range from 0 to 100, with values less than 70 representing pavements in acceptable condition. PDI values represent the overall condition of the pavement, based on the amount and severity of observable surface distress present in 500-foot monitoring segments within each section. It should be noted that monitoring segments are typically used for PDI purposes, since counting every crack, along with other distresses, over the entire length of each test section, could be very labor intensive.

The 500-foot monitoring segments used for this research study were selected soon after construction. For data consistency and reporting accuracy, the segments were not changed throughout the study. Therefore, the PDI values may not accurately reflect the performances over the entire 2600-foot length of the test/control sections, particularly due to the aforementioned discrepancies within the sections.

PDI surveys were conducted annually for seven years and a final survey was conducted in 2003, after ten years (see Table 13, below).

Table 13. Average Pavement Distress Index (PDI) per Test Section

TEST SECTIONS	1994 (Year 1)	1995 (Year 2)	1996 (Year 3)	1997 (Year 4)	1998 (Year 5)	1999 (Year 6)	2000 (Year 7)	2003 (Year 10)
Control Section: Virgin HMA Surface & Lower Course	0	0	7	7	7	27	47	55
Test Section 1: Virgin HMA Surface Course & Recycled CRM RAP Lower Course	0	0	7	7	7	13	42	56
Test Section 2: Recycled CRM RAP Surface & Lower Course	0	0	7	7	7	7	42	56
Test Section 3: Recycled CRM RAP Surface Course & Virgin HMA Lower Course	0	0	7	7	7	7	39	66

As shown in Table 13, all sections performed equally well for the first five years, with PDI values of seven after five years in service. After ten years in service, the PDI values indicate that all the sections performed similarly, though Test Section 3 (recycled CRM RAP surface course & virgin HMA lower course) performed slightly worse, with a PDI of 66.

PERFORMANCE EVALUATION SUMMARY

Overall, there was no clear indication of which section performed the best or the worst over the duration of the study. Although Test Section 3 (recycled CRM RAP surface course and virgin HMA lower course) had the highest PDI value after ten years, that was largely due to the amount and severity of longitudinal cracking within the 500-foot monitoring segment, and may not be truly representative of the overall performance of the entire 2600-foot test section, as the section had the least amount of transverse cracking.

The performance variations within the different sections indicate that factors other than pavement design (i.e. compaction of the underlying milled asphaltic pavement, the steep slopes and highly variable soils, consistency/quality of the rubberized asphaltic mix) may have impacted the overall performance of the pavement.

COSTS

The initial incorporation of crumb rubber into the binder in 1987 increased the cost of the binder by 76 percent; the rubberized asphalt binder was \$210.00 per ton, while the standard asphalt binder was \$119.00 per ton. Consequently, the cost of the CRM RAP mix was more expensive than the standard RAP mix used in 1987. Excluding the cost of milling the existing pavement, the standard RAP mix, with 2.4 percent standard asphalt binder added, was \$13.86 per ton. The cost of the CRM RAP mix, with the addition of 4.3 percent rubberized asphalt binder, was \$21.03 per ton, roughly 52 percent more expensive than the standard RAP mix.

The 1987 CRM RAP site was milled and the salvaged millings were reused in the 1993 overlay. The recycled CRM RAP mix, with 4.4 percent standard asphalt binder added, was 14 percent less expensive than the virgin HMA mix used in the control section, which had 6.2% standard asphalt binder added (\$20.82 per ton versus \$24.24 per ton, respectively). These costs do not include the cost to mill the 1987 project, since the pavement was scheduled for rehabilitation (including milling the existing pavement) anyway, and the use of the salvaged asphaltic materials was utilizing what would otherwise have been waste material. The variation in the costs was primarily due to the amount of binder required. Since the CRM RAP millings contained residual binder, less additional binder was required for that mix as opposed to the virgin HMA mix.

RESULTS OF WISDOT RESEARCH STUDY # 93-01a

This research study was able to successfully recycle a RAP pavement containing scrap tire rubber. In 1993, millings from a 1987 CRM RAP pavement were mixed with virgin aggregate and virgin binder to construct several test sections with recycled CRM RAP. A control section, consisting of a virgin HMA pavement, was also constructed. The main conclusions of this study are summarized on the following page.

1. The CRM RAP pavement was more resistant and more difficult to mill than a typical RAP pavement.
2. The recycled CRM RAP material had similar characteristics as standard RAP mixes and did not cause any significant construction difficulties.
3. The emission data showed that recycling a pavement that contains tire rubber does not appear to pose a threat to the health of the workers or to the environment.
4. Test results showed that the test and control sections performed similarly, and fairly well, over the duration of the study in regards to rutting, international roughness index (IRI), friction, and pavement distress index (PDI).
5. The initial addition of crumb rubber into the binder of the CRM RAP constructed in 1987 increased the cost of the binder by 76 percent. Resultantly, the cost per ton of the CRM RAP mix was 52 percent higher than the cost of the standard RAP mix. However, as expected for any RAP project, milling the CRM RAP pavement and recycling the millings into a new pavement reduced the project cost by 14 percent, in comparison with a virgin HMA pavement.
6. No individual test section or pavement design clearly outperformed all others. Thus, a CRM RAP can be successfully recycled and can perform similarly to a new hot mix asphalt pavement.

Once again, the inconsistent performances of the individual test sections (i.e. northbound lane vs. southbound lane, north half of section vs. south half of section, etc.) indicate that the pavement distress observed might not be due solely to the components of the test section, and may be caused by other factors, such as the compaction of the underlying milled asphaltic pavement, the steep slopes and highly variable soils, or the consistency/quality of the rubberized asphaltic mix.

OVERALL RESULTS & CONCLUSIONS

Two research studies were conducted to evaluate the effectiveness of crumb rubber modified (CRM) hot mix asphalt (HMA) pavements. The first study (# WI 89-04) evaluated the effectiveness of a rubberized asphalt binder mixed with a virgin aggregate gradation for use as an overlay and/or a stress absorbing interlayer, while the second study (# 93-01a) evaluated the recyclability of a reclaimed asphaltic pavement (RAP) containing tire rubber.

Numerous test sections and control sections were constructed for these studies for performance evaluations and comparisons. The test and control sections constructed consisted of the following pavement designs/configurations:

WisDOT Federal Experimental Project # WI 89-04

Test Section 1

2-inch Standard HMA Surface Course
3/8-inch CRM HMA Interlayer
3/4-inch to 1 1/2-inch Standard HMA Lower Course

Test Section 2

2-inch CRM HMA Surface Course
3/8-inch CRM HMA Interlayer
3/4-inch to 1 1/2-inch CRM HMA Lower Course

Test Section 3

1 1/4-inch CRM HMA Surface Course
1 1/4-inch to 2-inch CRM HMA Lower Course

Control Section

1 1/4-inch Standard HMA Surface Course
1 1/4-inch to 2-inch Standard HMA Lower Course

WisDOT Research Study # 93-01a

Control Section

1 1/2-inch Virgin HMA Surface Course
3-inch Virgin HMA Lower Course

Test Section 1

1 1/2-inch Virgin HMA Surface Course
3-inch Recycled CRM RAP Lower Course

Test Section 2

1 1/2-inch Recycled CRM RAP Surface Course
3-inch Recycled CRM RAP Lower Courses

Test Section 3

1 1/2-inch Recycled CRM RAP Surface Course
3-inch Virgin HMA Lower Course

The construction of the CRM HMA pavement and the recycled CRM RAP went fairly smoothly; only minor complications were encountered, primarily due to lack of experience with paving CRM asphaltic mixes, and were easily overcome. The emission data showed that recycling a

pavement that contains tire rubber does not appear to pose a threat to the health of the workers or to the environment.

No individual test or control section clearly outperformed all others. Test results showed that the pavement designs used for each study performed comparably with their respective control sections, over the duration of the studies, in regards to rutting, international roughness index (IRI), friction, and pavement distress index (PDI). The rubberized asphalt components of the pavement did not appear to increase or decrease the amount or rate of reflective cracking.

Thus, for study # WI 89-04, the inclusion of crumb rubber into a virgin HMA pavement for use as an overlay or as an interlayer did not enhance or impede the overall performance of the pavement, in comparison to a virgin HMA pavement. In addition, for study # 93-01a, the RAP mix containing tire rubber was successfully recycled and performed similarly to a new hot mix asphalt pavement.

The inclusion of crumb rubber, however, did increase the cost of both projects. The cost of rubberized asphalt binder was 1.8 to 3.7 times more expensive than the cost of standard asphalt cement binder. Accordingly, the CRM HMA and the CRM RAP mixes were 1.5 to 2.2 times more expensive than the standard HMA mix, respectively. Using the rubberized asphalt binder together with stone chips to create an interlayer also increased the project cost by approximately \$10,000 per lane-mile.

Cost savings were achieved when the CRM RAP was milled for recycling, as pavements constructed with reclaimed asphaltic pavement are generally less expensive than pavements constructed with virgin materials. The recycled CRM RAP mix was 14 percent less expensive than the virgin HMA mix.

At the time WisDOT initiated the two research studies included in this report, knowledge and experience with asphalt rubber was limited. This lack of expertise likely contributed to the mediocre performances of WisDOT's CRM asphaltic pavements. Since the early 1990's, there have been significant advances in asphalt rubber related technologies. Current research results

from other states show that CRM asphaltic pavements can perform better than standard HMA. It has been reported that the use of asphalt rubber has improved a pavement's resistance to reflective cracking, rutting, and oxidation; improved the ride quality; and reduced traffic noise. These positive findings have led several states, including Arizona, California, Florida, and Texas, to incorporate asphalt rubber into their standard specifications.

In January 2003, Caltrans published a guide explaining the current state-of-the-practice for asphalt rubber usage. This guide provided recommendations for the use of crumb rubber in asphaltic pavements, including design, materials, mixing procedures, and construction practices, based on years of research. According to the recommendations presented in the guide, several design and construction procedures used to construct WisDOT's CRM HMA pavements were identified as possible sources for the pavements' undistinguished performances. Some of the differences between the procedures used by WisDOT and the recommendations by Caltrans are shown in Table 14 below.

Table 14. WisDOT's Procedures Compared to Current Caltrans Recommendations

Design/Construction Practice	WisDOT Procedure	Caltrans Recommendation
Rubberized Asphalt Binder Content (of CRM HMA)	5.25-6.50%	7-10%
Extender Oil Content (of Rubberized Asphalt Binder)	0-3%	2.5-6.0%
Total Rubber Content (of Rubberized Asphalt Binder)	16-18%	18-22%
Scrap Rubber Content (of Rubberized Asphalt Binder)	81-100%	73-77%
Natural Rubber Content (of Rubberized Asphalt Binder)	0-19%	23-27%
Aggregate Gradation	varied	gap- or open-graded
Rubberized Asphalt Binder Mixing Temperature	325-375° F	375-425° F
CRM HMA Pavement Compaction Temperature	280° F	290° F (minimum)

Although other, primarily southern, states are reporting favorable results with CRM asphaltic pavements, they are also acknowledging increased costs. With performances similar to standard HMA pavements and higher costs, WisDOT's crumb rubber modified hot mix asphalt pavements have not proven to be cost-effective. As technology further advances, however, the use of crumb rubber in asphaltic pavements may become more cost-effective.

RECOMMENDATIONS

Based on the mediocre performances of WisDOT's CRM asphaltic pavements, at increased costs, it is recommended that WisDOT not use crumb rubber in new HMA pavements until further research results from other agencies, preferably with similar climatic conditions, prove the practice to be cost-effective. *

It is further recommended that WisDOT continue to track the research on asphalt rubber conducted by other agencies, as there is still a considerable need for finding alternate methods of disposal for waste tires. As energy production equipment ages and environmental restrictions tighten, there is no telling how much longer tires will continue to be burned in Wisconsin as a source of fuel. Thus, WisDOT should continue its search for proactive methods of minimizing or eliminating tire stockpiles in Wisconsin.

* WisDOT does not currently ban the use of any product, including crumb rubber, in warranted projects, provided they meet the warranty requirements. Thus, future use of crumb rubber in WisDOT's asphaltic pavements will likely be based on its cost-competitiveness in a free market.

APPENDIX A

STH 57 MIX DESIGN

ASPHALT RUBBER

2.1	Asphalt	Koch 200/300	81	%
	Extender	San Joaquin 1200S	3	%
	Rubber	Baker IGR-24	13	%
	Rubber	Tonson C-112	3	%
2.2	<u>Reacted Properties</u>			

Reaction Temperature = 350°F

Reaction Period = 90 min.

<u>Test Parameter</u>	<u>Result</u>	<u>Specification Limits</u>
Viscosity, 350F	3300 cp	1500-4000 cp
Cone Penetration	97	80-150
Softening Point	133°F	110°F min.
Resilience, ²⁰ 60 sec recovery; %	14%	0% min.

Laboratory Mixture Fabrication

Mixing Temperature	<u>300°F</u>
Compaction Temperature	<u>275°F</u>
Compaction Effort	<u>50/50 blow Marshall</u>

<u>Design Binder Content</u>	(by total mix weight)	<u>5.25% +/- 0.2%</u>
	(by aggregate weight)	<u>5.5% +/- 0.2%</u>

Recommended Mix Temperature 300°F - 325°F

Minimum Laydown Temperature 280°F

Laboratory Density at Design Binder Content 155.5 pcf

Field Compacted Density 150.8 pcf min. (97% min)



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Lab Report No. I-89-146

Work Order No. _____

Report Date 6/6/89

Material Ground Rubber

Identification Baker IGR-24, Lot # 9638

Source Baker Rubber, South Bend Indiana

Sampled By _____ Date _____ Tested By Mark Lively

Requested By Bob Smith Date _____ Reviewed By [Signature]

Test Procedure _____

RESULTS in % Passing

<u>Sieve Size</u>	<u>Lot # 9638</u>	<u>ISI Dense Graded Limits</u>
#8	100	100
#10	100	100
#16	99.8*	100
#20	99.0	—
#30	79.4	70-100
#40	48.6	—
#80	10.2	0-20
#200	0.3	0-5
Moisture Content	0.06	

*Rubber very slightly off spec.



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Lab Report No. 1-89-32

Work Order No. _____

Report Date 3/13/89

Material Ground Rubber

Identification Tonson C-112

Source Tonson Rubber, Chandler Az.

Sampled By Mark Lively/Crafco Date _____ Tested By P. Petroff

Requested By J. Chehovits Date _____ Reviewed By JL

Test Procedure Rubber Gradation

RESULTS

<u>SIEVE SIZE</u>	<u>C112</u> <u>% Passing</u>
#10	100
#16	98.4
#30	43.1
#40	23.3
#50	11.1
#80	2.6
#100	1.4
#200	0.2



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Lab Report No. I-90-175

Work Order No. I-145/057

Report Date 6/4/90

Material Asphalt Rubber Hot Mix Binder

Identification Wisconsin using Koch 200/300 AC @ 81%, San Joaquin 1200 oil @ 3%,
Baker IGR-24 rubber @ 13% and Tonson C112 @ 3%.

Source Lab Stock

Sampled By _____ Date _____ Tested By Petroff/Hobbs

Requested By Anne Stonex Date 5/7/90 Reviewed By [Signature]

Test Procedure 24 hr reaction @ 350°F in oven.

RESULTS in minutes reaction

Test Performed	30	60	90	120	360	1440
Viscosity, Brookfield @ 350 °F						
Probe # <u>3</u> RPM <u>20</u> in centipoise	2800	3300	3300	3300	3500	2300
Viscosity, Brookfield @ 350 °F in centipoise	2500	3000	3000	2800	2300	1800
Penetration, Cone @ 77°F in 1/10 mm, 150 g, 5 sec.	--	--	97	--	106	107
Penetration, Needle @ 77°F in 1/10mm, 100 g, 5 sec.	--	--	109	--	122	122
Penetration, Needle @ 39.2°F in 1/10mm, 200 g, 60 sec.	--	--	60	--	61	59
Resilience @ 77°F in % rebound	--	--	14	--	9	6
Ductility @ 39.2°F @ 1 cm per min. Rate, in cm pulled @ failure	--	--	17	--	18	19
Ductility @ 77°F @ 5cm/min. in cm	--	--	30	--	30	31
Softening 1 in in °F	136	135	133	132	127	125
Fracture Temperature						
°F Lowest Passing	--	--	6	--	6	6
°F Fracture	--	--	4	--	4	4

Asphalt Penetration @ 77°F was 242
Asphalt Softening Point was 100°F



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(602) 268-0674
Watts. 800-526-4548

Lab Report No. I-90-159

Work Order No. I-145/057

Report Date 5/14/90

Material Asphalt Rubber Hot Mix Binder

Identification Koch 200/300 @ 81.2%, SJ 1200 oil @ 2.8%, IGR-24 @ 13.2% and C112

Source Koch of Minneapolis, MN rubber @ 28%

Sampled By _____ Date _____ Tested By Petroff/Hobbs

Requested By Anne Stoner Date 5/9 Reviewed By JHR

Test Procedure React for 90 min. @ 350°F in oven

RESULTS @ 90 minutes reaction

<u>Test Performed</u>	<u>90</u>
Viscosity, Brookfield @ <u>350</u> °F	
Probe # <u>3</u> RPM <u>20</u> in centipoise	3300
Viscosity, Brookfield @ <u>350</u> °F in centipoise	3000
Penetration, Cone @ 77°F in 1/10 mm, 150 g, 5 sec.	94
Penetration, Needle @ 77°F in 1/10mm, 100 g, 5 sec.	116
Penetration, Needle @ 39.2°F in 1/10mm, 200 g, 60 sec.	55
Resilience @ 77°F in % rebound	7
Ductility @ 39.2°F @ 1 cm per min. Rate, in cm pulled @ failure	20
Softening 1 in in °F	132
Fracture Temperature	
°F Lowest Passing	10
°F Fracture	8

Asphalt Penetration @ 77°F = 242
Asphalt Softening Point in °F = 100



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Lab Report No. I-90-156

Work Order No. I-145/057

Report Date 5/14/90

Material Asphalt Rubber Hot Mix Binder

Identification Koch 200/300 @ 83%, Baker IGR-24 @ 17% for Wisconsin

Source Koch of Minneapolis, MN

Sampled By _____ Date _____ Tested By P. Petroff

Requested By Anne Sronex Date _____ Reviewed By [Signature]

Test Procedure React @ 350°F for 90 min. in oven

RESULTS @ 90 minutes reaction

Test Performed

Viscosity, Brookfield @ <u>350</u> °F	
Probe # <u>3</u> RPM <u>20</u> in centipoise	4000
Viscosity, Hake @ <u>350</u> °F in centipoise	3300
Penetration, Cone @ 77°F in 1/10 mm, 150 g, 5 sec.	74
Penetration, Needle @ 77°F in 1/10mm, 100 g, 5 sec.	84
Penetration, Needle @ 39.2°F in 1/10mm, 200 g, 60 sec.	44
Resilience @ 77°F in % rebound	26
Ductility @ 39.2°F @ 1 cm per min. Rate, in cm pulled @ failure	17
Softening 1 in. in °F	137
Fracture Temperature	
°F Lowest Passing	16
°F Fracture	14

Asphalt Penetration @ 77°F = 242
Softening Point in °F = 100



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Lab Report No. I-90-186
Work Order No. I-145/057
Report Date 5/31/90

Material Asphalt Rubber Hot Mix
Identification Wisconsin, Green Bay Project
Source Lab Stock
Sampled By _____ Date _____ Tested By Paul Petroff
Requested By Anne Stonex Date _____ Reviewed By [Signature]
Test Procedure Moisture Resistance Testing of Compacted Asphalt Rubber
Specimens

RESULTS

Aggregate: Sheboggan Sand & Gravel Crushed Fines (7/16") 76
Sheboggan Sand & Gravel 5/8" coarse
aggregate 24
Binder Koch 200/300 @ 81%, San Joaquin 1200S @ 3%,
Baker IGR-24 @ 13%, Tonson C112 @ 3% 5.25*

Mixture Maximum SG 2.5676 Mix Temp. 300°F Compact Temp 275°F
Compaction Effort 12 Blows per side

UNCONDITIONED SPECIMEN			Tensile
No.	Bulk SG	Air Voids	Strength, psi
<u>1</u>	<u>2.4247</u>	<u>5.6%</u>	<u>55.4</u>
<u>2</u>	<u>2.4077</u>	<u>6.2%</u>	<u>56.6</u>
<u>Ave</u>	<u>2.4162</u>	<u>5.9%</u>	<u>56.0</u>

Note: Conditioning is vacuum saturation followed by 0°F freeze for 16 hours and then 24 hour 140°F water bath soak. Specimens are conditioned to 77°F prior to testing tensile strength.

MOISTURE CONDITIONED SPECIMENS			Void	% Void	Tensile
No.	Bulk SG	Air Voids	Volume, cc	Saturation	Strength, psi
<u>3</u>	<u>2.4193</u>	<u>5.8</u>	<u>29.7</u>	<u>71.0%</u>	<u>50.1</u>
<u>4</u>	<u>2.4143</u>	<u>6.0</u>	<u>31.3</u>	<u>72.5%</u>	<u>47.0</u>
<u>Ave</u>	<u>2.4168</u>	<u>5.9</u>		<u>71.7%</u>	<u>48.6</u>

TENSILE STRENGTH RATIO 86.7%

*By total mix weight.



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PROJECT I-145/057

DATE 5/29/90

MIX DESIGN DATA SUMMARY

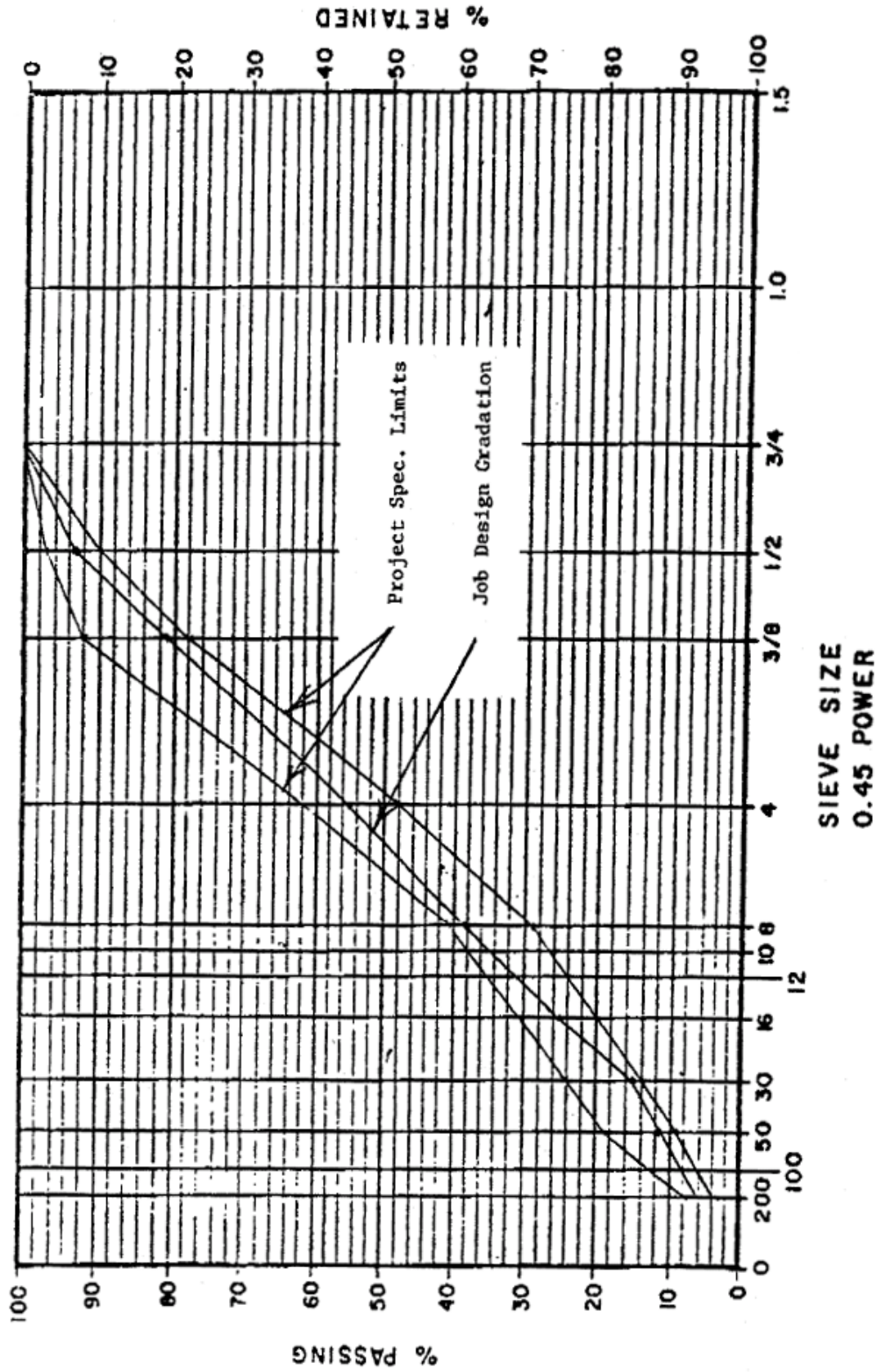
AGGREGATE

	<u>MATERIAL</u>	<u>SOURCE</u>	<u>DESIGNATION</u>	<u>% BLEND</u>
1.	Coarse Aggregate	Sheboggan S & G	Coarse 5/8"	24
2.	Crushed Stone	" "	7/16" minus	76
3.				
4.				

GRADATION ANALYSIS

<u>BLEND %</u>		24	76			<u>JOB MIX FORMULA</u>	<u>SPECIFICATION RANGE</u>
<u>MATERIAL</u>		1	2	3	4		
<u>SIEVE SIZE</u>	3/4	100	100			100	100
	1/2	70.6	100			92.9	90-97
	3/8	22.8	99.2			80.8	78-92
	4	2.2	71.1			54.6	48-62
	8	1.6	50.2			38.5	29-41
	16	1.5	32.8			25.3	---
	30	1.4	19.7			15.3	14-24
	50	1.2	14.6			11.4	9-19
	200	1.1	6.8			5.4	3-7
<u>BULK SP. GR.</u>		2.7575	2.7489			2.7509	
<u>APPARENT SP. GR.</u>		2.8218	2.8230			2.8227	
<u>% ABSORPTION</u>		0.8	1.0			0.9	

Project I-145/057



Project: Wisconsin
 Date: 5/31/90
 WO # I-145/057

MIXTURE DESIGN DATA SUMMARY

Binder # Content %	Bulk Sp. Gr.	Max. Sp. Gr.	Density (pcf)	Air Voids %	V.M.A.	Effective Binder %	Stability (pounds)	Flow (1/100 inch)
5.25	2.4920	2.5676	155.5	3.0%	14.2%	4.7%	1836	16
6.25	2.4850	2.5291	155.1	1.8%	15.3%	5.7%	1824	19
7.0	2.4647	2.5010	153.8	1.5%	16.7	6.5%	1651	20
7.75	2.4519	2.4733	153.0	0.9%	17.8%	7.2%	1491	20

* Note: Binder content is by total mixture weight.

NOTES: Specimens are mixed at 300°F. Compaction is at 275°F using 50/50 blow Marshall (Hand held).

Asphalt rubber binder is as shown on report I-90-175

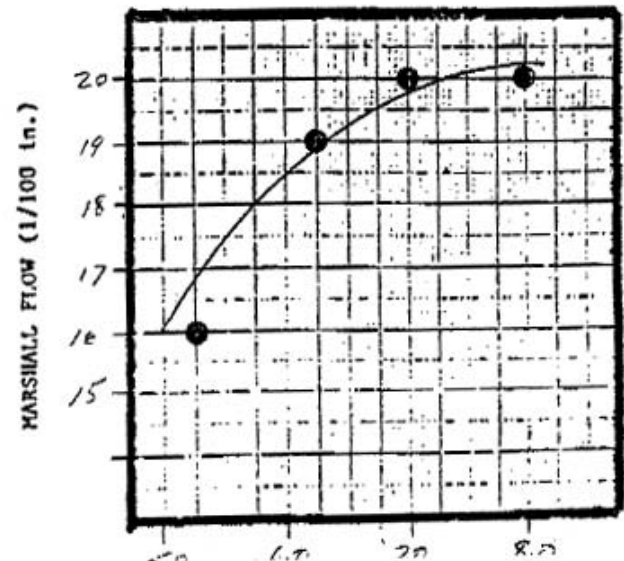
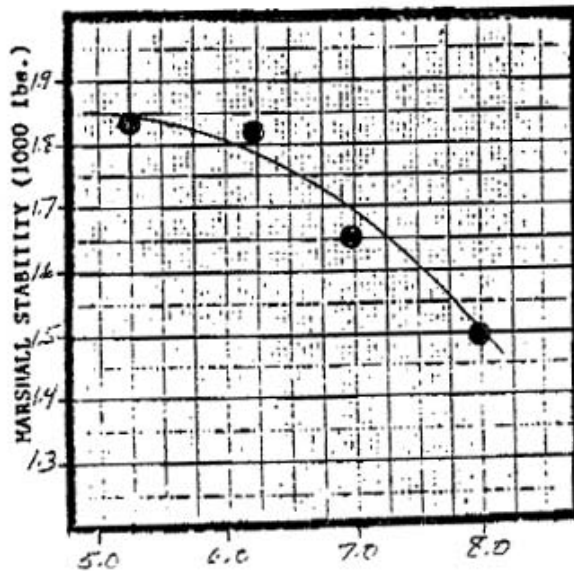
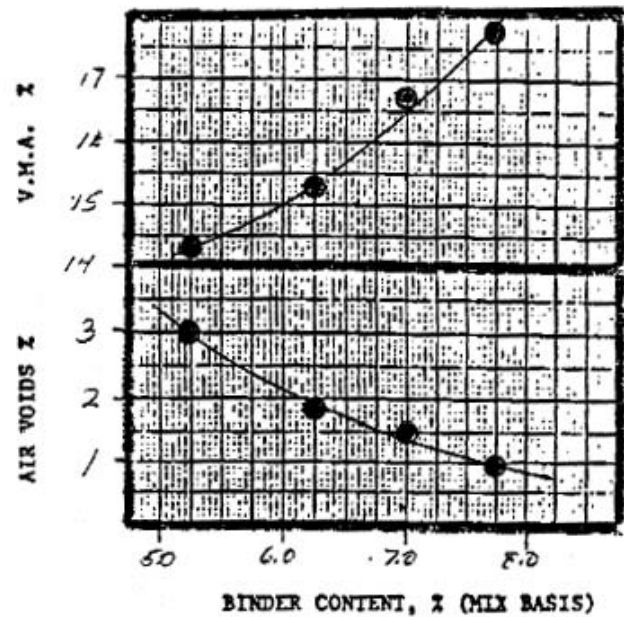
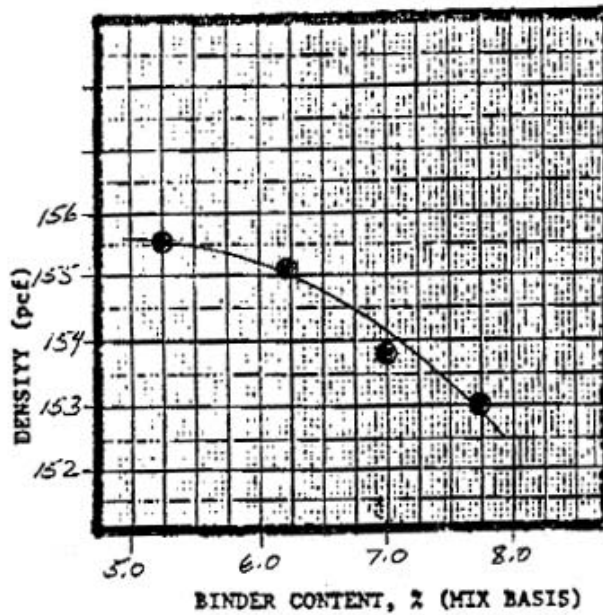


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PROJECT I-145/057

DATE 6/1/90

MIXTURE DESIGN DATA PLOTS



APPENDIX B

STH 35 MIX DESIGN



MATHY CONSTRUCTION CO.

GENERAL CONTRACTORS

915 COMMERCIAL COURT • POST OFFICE BOX 189 • ONALASKA, WISCONSIN 54650

PHONE 608-783-6411 • FAX 608-783-4311

250-2186-93

Report of Bituminous Mix Design

Test #.....: 8059-MVS
 Date.....: July 12, 1993
 Project Number.....: 5162-03-71
 Project Name.....: DESOTO-GENOA ROAD
 County.....: CRAWFORD & VERNON
 Specifications.....: MV GRADE 2 AND 3

AGGREGATE SOURCES

Percent	Material	Supplier\Source	SpG
1: 36.00	: LIMESTONE	: PEDRETTI 27,13,7 VERNON	: 2.566
2: 20.00	: RAP/RUBBER PEN -35	: BLENDED PEN-93	: 2.717
3: 20.00	: BLEND SAND	: FRANZ 31,11,7 CRAWFORD	: 2.642
4: 24.00	: BLEND SAND	: PRAIRIE 12,7,7 CRAWFORD	: 2.665
5:	: VIRGIN AGGREGATE	: BLEND	:
Total: 100.00	Effec SpG: 2.672	SpG Total: 2.635	
Rap Blend=: 45% # 1;25% # 3;30% # 4;FOR VIRGIN AGGREGATE 5.5%AC IN RAP			

AGGREGATE GRADATIONS

Gradations						
	#1	#2	#3	#4	#5	Job Mix Spec.
1" :	100.0 :	100.0 :	100.0 :	100.0 :	100.0 :	100.0 :
3/4 :	100.0 :	100.0 :	100.0 :	100.0 :	100.0 :	100.0 :
1/2 :	82.0 :	100.0 :	100.0 :	100.0 :	92.0 :	93.5 :
3/8 :	43.0 :	98.0 :	100.0 :	100.0 :	74.0 :	79.1 :
# 4 :	5.1 :	81.0 :	100.0 :	99.0 :	57.0 :	61.8 :
# 8 :	3.6 :	63.0 :	100.0 :	89.0 :	53.0 :	55.3 :
# 16 :	3.3 :	52.0 :	100.0 :	73.0 :	48.0 :	49.1 :
# 30 :	3.3 :	42.0 :	98.0 :	45.0 :	39.0 :	40.0 :
# 50 :	3.3 :	29.0 :	62.0 :	5.0 :	18.0 :	20.6 :
#100 :	:	:	:	:	:	:
#200 :	2.6 :	14.0 :	1.9 :	0.1 :	1.7 :	4.1 :
PI :	NP :	NP :	NP :	NP :	:	:
Crush :	100 :	:	0 :	0 :	:	:
Thin						
& :	0 :		0 :	0 :		
Elong						



MATHY CONSTRUCTION CO.

GENERAL CONTRACTORS

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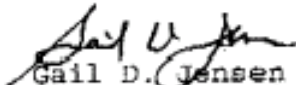
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Report of Bituminous Mix Design
 Test #.....: 8059-MVS
 Date.....: July 12, 1993
 Project Number...: 5162-03-71
 Project Name.....: DESOTO-GENOA ROAD
 County.....: CRAWFORD & VERNON
 Specification....: MV GRADE 2 AND 3

MIX PROPERTIES

	#1	#2	#3	#4	#5
AC Content % By Weight :	4.50	5.00	5.50	6.00	6.50
Rice SpG.....:	2.491	2.473	2.455	2.437	2.419
Air Voids %.....:	6.0	4.9	3.4	1.9	1.2
Density Lbs @ 77 F.Bulk:	145.7	146.4	147.5	148.8	148.8
Density SpG Bulk.....:	2.341	2.352	2.370	3.391	2.391
Stability @ 140 Deg F :	1247	1358	1438	1272	1217
Flow 0.01 In.....:	7.0	7.5	8.5	9.3	10.3
VMA.....:	15.2	15.2	15.0	14.7	15.2
Asphaltic Material.....:	120.150	Voids Total Mix : 3.4			
Asphalt Source.....:	KOCH	VMA.....: 15.0			
Specific Gravity.....:	1.028	Added AC.. : 4.4			
Recommended AC Binder..:	5.5	Added AC.. : 4.4			
Recommended AC Surface :	5.5	Dry Back %.. : 0.0038			
Density Bulk SpG.....:	2.370	Stability.. : 1438			
Density lbs/cu ft.Bulk..:	147.5	Flow..... : 8.5			
Density Max SpG.....:	2.455				
TSR @ Blow Count.....:	95.4 @ 15				
Blow/Side (Beveled)....:	50				
Mix Temp/Degrees F.....:	270-300				

*Since this design is material specific, the conclusions and recommendations contained within are obtained from material submitted to and subjected to observations under laboratory conditions. Adjustments may become necessary when field laboratory data is obtained from plant produced mix. No guarantee or warranty is implied or offered.


 Gail D. Jensen P.E.
 Materials Engineer